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ACI
PROCEEDINGS
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ACI JOURNAL

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Coming next month in the JOURNAL

Theoretical and practical aspects of winter concreting will be presented by E. G. SWENSON in the November JOURNAL in his article, "Winter Concreting Trends in Europe." This paper was originally presented at a panel on winter concreting at the ACI Regional Meeting in Montreal in October, 1956.

•

G. W. WASHA and J. C. SAEMANN are co-authors of the paper, "Variation of Mortar and Concrete Properties with Temperature," a detailed study of the effects of temperature upon strength, stiffness, and toughness of mortars and concretes. Test temperatures varied between −70 and 450 F.

•

"A Critical Look at Slab Design Methods" prepared by K. E. McKEE and E. I. FIESENHEISER will include graphs of maximum moments for different edge conditions, over the full range of variation of side length ratios. Consideration is limited to uniformly loaded rectangular slabs supported on all four sides.

•

Test data obtained in several investigations of the relation between ultimate bearing capacity and the ratio of footing area to the loaded area of concrete and rock foundations are summarized in "Bearing Capacity of Concrete" by WILLIAM SHELSON.

•

SAM HASSID briefly explains a method which has been found useful in analyzing frames subjected to temperature changes, in a paper entitled "Temperature Stresses in Continuous Frames."

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members Without Shear Reinforcement*

By CHARLES S. WHITNEY†

SYNOPSIS

The method for estimating shear strength proposed in this paper is radically different from that currently in use, but it appears to be well supported by the results of tests which have covered a wide range of proportions and concrete and steel strengths.

The conventional shear formula

$$v = V/bjd = k f_c'$$

is not suitable for use because the shear strength is not a simple function of concrete strength, but depends largely on the amount of flexural reinforcement and its efficiency. Also tests show that because of bond failure due to splitting of concrete, the flexural reinforcement cannot be fully effective if it is too closely spaced.

FLAT SLABS AND FOOTINGS

This method for the ultimate strength design for shear of flat slabs and column footings is based on an analysis of the University of Illinois and Portland Cement Association tests of flat slabs[1] and the University of Illinois tests of column footings.[2,3]

Basic tests

The most important series of tests on column footings were made at the University of Illinois,[2,3] sponsored by the committee on reinforced concrete research of the American Iron and Steel Institute and reported by Frank E. Richart in 1948. There were tested to destruction 132 footings, for the most part 7 ft square with their depth d varying from 8 to 16 in. They were reinforced with bars of various types of deformation, including the new type (ASTM A 305). Steel yield strengths varied from 44,000 to 82,900 psi and concrete strengths from about 2000 to 5000 psi (test data in Table 1).

At the request of the joint committee on shear and diagonal tension, the Reinforced Concrete Research Council sponsored a series of tests on flat slabs carried out at the University of Illinois[1] in 1952. This consisted of 24 slabs, 6 ft square and 6 in. thick, with steel ratios of 0.0055 to 0.037, and with f_c'

*Received by the Institute May 27, 1957. To be presented at the ACI 54th annual convention, Chicago, Ill., Feb. 26, 1958. Title No. 54-15 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 4, Oct. 1957, *Proceedings* V. 54. Separate prints are available at 75 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Jan. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit, 19, Mich.

†Member American Concrete Institute, Partner, Ammann & Whitney, Consulting Engineers, New York, N. Y. and Milwaukee, Wis.

TABLE 1—RESULTS OF FOOTING

Test specimen No.		p, percent	d, in.	f_c', psi	f_s, ksi	P_{test}, kips	* P_{yield}, kips	P_{flex}, kips	P_{calc}, kips	$\frac{P_{test}}{P_{flex}}$	$\frac{P_{yield}}{P_{calc}}$	$\frac{P_{test}}{P_{calc}}$
Footings—Series 1 (a =												
105	a	0.393	14	3430	66.8	524	420	570	471.5	0.920	0.891	1.111
	b	0.393	14	2390	66.8	404	360	560	466.2	0.721	0.772	0.867
106	a	0.391	14	3750	62.4	525	425	533	455.6	0.985	0.933	1.152
	b	0.391	14	3620	62.4	474	425	533	454.2	0.890	0.936	1.044
107	a	0.395	14	3620	59.8	474	350	517	445.0	0.917	0.787	1.065
	b	0.395	14	3410	59.8	425	360	515	443.6	0.825	0.812	0.958
108	a	0.374	14	4160	52.7	475	360	435	393.3	1.091	0.915	1.208
	b	0.374	14	3275	52.7	450	330	432	401.3	1.042	0.822	1.121
109	a	0.561	14	2965	76.9	538	(538)	904	643.4	0.595	0.836	0.836
	b	0.561	14	3080	76.9	460	(460)	906	646.1	0.507	0.712	0.712
110	a	0.561	14	3210	70.9	500	(500)	843	611.7	0.593	0.817	0.817
	b	0.561	14	2700	70.9	520	490	832	607.6	0.625	0.806	0.856
111	a	0.632	14	3010	61.6	480	(480)	823	602.3	0.583	0.797	0.797
	b	0.632	14	3400	61.6	573	510	830	607.6	0.690	0.839	0.943
112	a	0.748	14	3470	74.0	480	(480)	1151	767.8	0.417	0.625	0.625
	b	0.748	14	2815	74.0	520	(520)	1130	753.1	0.460	0.690	0.690
109R	a	0.561	14	4070	66.1	575	550	800	593.2	0.719	0.927	0.969
	b	0.561	14	4180	66.1	550	(550)	801	594.4	0.687	0.925	0.925
110R	a	0.561	14	3160	74.7	508	(508)	884	632.9	0.575	0.803	0.803
	b	0.561	14	3500	74.7	593	(593)	890	636.7	0.666	0.931	0.931
Footings—Series 2 (a =												
201	a	1.000	14	2660	62.4	298	(298)	635	423.9	0.470	0.703	0.703
	b	1.000	14	2665	62.4	340	(340)	635	424.6	0.535	0.801	0.801
202	a	0.694	14	2385	62.4	419	(419)	654	470.5	0.641	0.891	0.891
	b	0.694	14	2180	62.4	400	(400)	648	450.7	0.617	0.888	0.888
203	a	0.510	14	2630	62.4	420	(420)	675	527.0	0.622	0.797	0.797
	b	0.510	14	2020	62.4	380	(380)	662	519.1	0.575	0.732	0.732
204	a	1.032	12	2580	62.4	400	(400)	935	607.1	0.428	0.659	0.659
	b	1.032	12	2540	62.4	400	(400)	933	606.0	0.429	0.660	0.660
205	a	0.748	14	2260	62.4	460	(460)	946	660.5	0.486	0.696	0.696
	b	0.748	14	2405	62.4	460	(460)	952	665.8	0.483	0.691	0.691
206	a	0.580	16	2920	62.4	520	(520)	1002	748.6	0.519	0.695	0.695
	b	0.580	16	2680	62.4	610	(610)	996	745.3	0.612	0.818	0.818
207	a	1.250	8	4250	62.4	360	(360)	522	343.3	0.690	1.049	1.049
	b	1.250	8	4040	62.4	342	(342)	520	341.0	0.658	1.003	1.003
208	a	0.786	10	4010	62.4	357	350	530	375.2	0.674	0.933	0.951
	b	0.786	10	3845	62.4	380	370	529	375.2	0.718	0.986	1.013
209	a	0.536	12	2890	62.4	440	390	523	409.3	0.841	0.953	1.075
	b	0.536	12	2355	62.4	420	380	515	400.1	0.816	0.950	1.050
210	a	1.000	10	4320	62.4	450	(450)	667	442.7	0.675	1.016	1.016
	b	1.000	10	4230	62.4	400	(400)	666	440.3	0.601	0.908	0.908
211	a	0.694	12	3660	62.4	480	460	676	485.0	0.710	0.948	0.990
	b	0.694	12	4270	62.4	520	480	682	488.0	0.762	0.984	1.066
212	a	0.510	14	3885	62.4	520	500	689	536.2	0.755	0.932	0.970
	b	0.510	14	4100	62.4	500	(500)	691	537.6	0.723	0.930	0.930
213	a	1.250	8	4500	62.4	340	(340)	525	344.4	0.648	0.987	0.987
	b	1.250	8	4550	62.4	340	(340)	526	345.1	0.646	0.985	0.985
214	a	0.786	10	4775	62.4	460	420	536	379.1	0.858	1.108	1.213
	b	0.786	10	4940	62.4	479	400	537	379.9	0.892	1.053	1.261
215	a	0.536	12	5045	62.4	480	390	536	414.6	0.896	0.941	1.158
	b	0.536	12	4230	62.4	480	400	533	413.5	0.901	0.967	1.161
216	a	1.250	8	4550	62.4	360	(360)	526	345.1	0.684	1.043	1.043
	b	1.250	8	4470	62.4	360	(360)	525	343.9	0.686	1.047	1.047
217	a	0.786	10	3720	62.4	380	(380)	528	374.8	0.720	1.016	1.016
	b	0.786	10	4735	62.4	460	400	536	378.4	0.858	1.057	1.216
218	a	0.536	12	4140	62.4	480	380	532	411.4	0.902	0.924	1.167
	b	0.536	12	4430	62.4	440	400	534	412.5	0.824	0.970	1.067

NOTE: In the expression $s\ f_c'/3000$, s is spacing, center to center, expressed in terms of bar diameters.
*If no P_{yield} was reported, P_{test} is stated in parentheses.

TESTS REPORTED EARLIER[2]

$\frac{P_{calc}}{P_{flex}}$	$\frac{M_u}{d^2}\sqrt{\frac{d}{l}}$, psi	Reinforcement: No. one way	Size, #	Hooks	Spacing, ϕ	l_h, ϕ	T, kips	u, psi	$\frac{u}{f_c'}$	$s\,\frac{f_c'}{3000}$	v, psi
84 in., r = 14 in., l = 17.5 in.)											
0.827	223	42	3	H	5.2	50.7	7.33	364	0.106	5.95	298
0.833	219	42	3	H	5.2	50.7	7.33	284	0.119	4.14	228
0.855	211	23	4	H	7.2	38.0	12.50	482	0.128	9.00	296
0.852	210	23	4	H	7.2	38.0	12.50	437	0.121	8.69	268
0.861	203	15	5	H	9.1	30.4	18.50	528	0.146	10.86	261
0.861	202	15	5	H	9.1	30.4	18.50	475	0.139	10.23	241
0.904	171	10	6	H	11.7	25.4	23.20	626	0.150	16.09	269
0.929	170	10	6	H	11.7	25.4	23.20	582	0.178	12.66	254
0.712	353	60	3	H	3.6	50.7	8.45	316	0.106	3.56	304
0.713	355	60	3	H	3.6	50.7	8.45	269	0.087	3.70	261
0.726	329	33	4	H	5.0	38.0	14.20	389	0.121	5.24	283
0.730	326	33	4	H	5.0	38.0	14.20	407	0.151	4.41	294
0.732	322	24	5	H	5.5	30.4	19.10	408	0.136	5.52	272
0.732	326	24	5	H	5.5	30.4	19.10	483	0.142	6.23	324
0.667	447	20	6	H	5.6	25.4	32.60	455	0.131	6.36	272
0.666	436	20	6	H	5.6	25.4	32.60	502	0.179	5.16	294
0.742	315	60	3	H	3.6	50.7	7.28	315	0.077	4.88	323
0.742	316	60	3	H	3.6	50.7	7.28	301	0.072	5.02	311
0.716	345	33	4	H	5.0	38.0	14.90	401	0.127	5.16	287
0.715	348	33	4	H	5.0	38.0	14.90	465	0.133	5.72	337
84 in., r = 14 in., l = 17.5 in.)											
0.668	407	42	4	H	3.9	46.0	12.50	243	0.091	3.46	285
0.669	408	42	4	H	3.9	46.0	12.50	277	0.104	3.46	326
0.719	321	35	4	H	4.7	42.0	12.50	338	0.142	3.74	303
0.696	316	35	4	H	4.7	42.0	12.50	337	0.155	3.42	289
0.781	265	30	4	H	5.5	38	12.5	334	0.127	4.82	238
0.784	259	30	4	H	5.5	38	12.5	306	0.151	3.72	216
0.649	453	52	4	S	3.1	42	12.5	249	0.097	2.67	289
0.650	452	52	4	S	3.1	42	12.5	250	0.098	2.62	289
0.698	366	44	4	S	3.7	38	12.5	291	0.129	2.79	261
0.699	370	44	4	S	3.7	38	12.5	289	0.120	2.97	261
0.747	320	39	4	S	4.2	34	12.5	325	0.111	4.09	236
0.748	318	39	4	S	4.2	34	12.5	382	0.143	3.75	277
0.658	472	42	4	H	3.9	50	12.5	334	0.079	5.53	475
0.656	468	42	4	H	3.9	50	12.5	319	0.079	5.25	452
0.708	345	33	4	H	5.0	46	12.5	329	0.082	6.55	342
0.709	344	33	4	H	5.0	46	12.5	350	0.091	6.28	365
0.783	262	27	4	H	6.1	42	12.5	407	0.141	5.88	319
0.777	253	27	4	H	6.1	42	12.5	397	0.168	4.79	303
0.664	431	42	4	S	3.9	46	12.5	359	0.083	5.62	431
0.661	428	42	4	S	3.9	46	12.5	314	0.074	5.50	383
0.717	335	35	4	S	4.7	42	12.5	375	0.102	5.73	346
0.716	338	35	4	S	4.7	42	12.5	403	0.094	6.69	375
0.778	272	30	4	S	5.5	38	12.5	406	0.104	7.12	294
0.778	273	30	4	S	5.5	38	12.5	389	0.095	7.52	283
0.656	474	42	4	H	3.9	50	12.5	314	0.070	5.85	449
0.656	475	42	4	H	3.9	50	12.5	313	0.069	5.92	449
0.707	350	33	4	H	5.0	46	12.5	419	0.088	7.80	438
0.707	351	33	4	H	5.0	46	12.5	437	0.089	8.07	460
0.774	267	27	4	H	6.1	42	12.5	438	0.087	10.26	347
0.776	266	27	4	H	6.1	42	12.5	440	0.104	8.60	347
0.656	475	42	4	S	3.9	50	12.5	332	0.073	5.92	475
0.655	473	42	4	S	3.9	50	12.5	333	0.075	5.81	475
0.511	344	33	4	S	5.0	46	12.5	351	0.094	6.08	363
0.706	349	33	4	S	5.0	46	12.5	420	0.089	7.73	440
0.773	264	27	4	S	6.1	42	12.5	442	0.107	8.42	346
0.772	265	27	4	S	6.1	42	12.5	404	0.091	9.01	318

Table 1 continues on following pages.

TABLE 1 (Cont.)—RESULTS OF FOOTING

Test specimen No.		p, percent	d, in.	f_c', psi	f_s, ksi	P_{test}, kips	P_{yield}*, kips	P_{flex}, kips	P_{calc}, kips	$\frac{P_{test}}{P_{flex}}$	$\frac{P_{yield}}{P_{calc}}$	$\frac{P_{test}}{P_{calc}}$
Footings—Series 3 (a =												
305	a	0.393	14	3520	66.1	560	430	565	475.3	0.991	0.905	1.178
	b	0.393	14	3670	66.1	558	440	566	476.8	0.986	0.923	1.170
307	a	0.391	14	3320	74.7	520	425	631	504.4	0.824	0.843	1.031
	b	0.391	14	3810	74.7	554	470	635	507.1	0.872	0.927	1.092
308	a	0.391	14	3820	60.9	536	470	521	448.9	1.029	1.047	1.194
	b	0.391	14	3730	60.9	450	360	521	447.7	0.864	0.804	1.005
309	a	0.391	14	3870	62.4	474	410	534	455.6	0.888	0.890	1.040
	b	0.391	14	3570	62.4	450	440	532	454.2	0.846	0.969	0.991
310	a	0.391	14	4110	74.7	600	450	636	507.1	0.943	0.887	1.183
	b	0.391	14	3920	74.7	525	450	635	507.1	0.827	0.887	1.035
312	a	0.395	14	3630	63.8	512	410	550	462.1	0.931	0.887	1.108
	b	0.395	14	3260	63.8	375	370	548	460.9	0.684	0.803	0.814
314	a	0.395	14	3200	70.6	550	520	603	488.6	0.912	1.064	1.126
	b	0.395	14	3880	70.6	465	(465)	608	492.6	0.765	0.944	0.944
315	a	0.395	14	2880	53.2	375	350	458	415.9	0.819	0.842	0.902
	b	0.395	14	3260	53.2	461	375	460	417.1	1.002	0.899	1.105
316	a	0.395	14	3900	59.8	525	375	518	445.0	1.013	0.843	1.180
	b	0.395	14	4320	59.8	500	380	519	447.7	0.964	0.849	1.117
317	a	0.395	14	3610	63.8	486	375	550	462.1	0.884	0.812	1.052
	b	0.395	14	3720	63.8	520	380	550	462.1	0.945	0.822	1.125
319	a	0.395	14	3800	70.6	475	460	607	491.2	0.783	0.936	0.967
	b	0.395	14	3500	70.6	540	460	605	490.0	0.893	0.939	1.102
321	a	0.374	14	3650	69.2	485	440	564	471.5	0.860	0.933	1.029
	b	0.374	14	3830	69.2	516	420	565	472.7	0.913	0.889	1.092
323	a	0.374	14	3360	66.5	488	360	541	458.2	0.902	0.786	1.065
	b	0.374	14	3760	66.5	511	340	543	459.5	0.941	0.740	1.112
324	a	0.374	14	3890	73.0	494	425	595	487.3	0.830	0.872	1.014
	b	0.374	14	4010	73.0	504	400	596	487.3	0.845	0.821	1.034
326	a	0.374	14	3070	69.2	512	(512)	561	468.8	0.913	1.092	1.092
	b	0.374	14	3820	69.2	519	410	565	472.7	0.918	0.867	1.098
Footings—Series 4 (a =												
403	a	0.395	14	3435	58.9	440	350	516	439.7	0.853	0.796	1.001
	b	0.395	14	1960	58.9	360	330	502	431.8	0.717	0.764	0.834
Footings—Series 7 (a =												
701	a	0.982	21	3860	60.9	434	350	502	411.1	0.865	0.851	1.056
	b	0.982	21	3545	60.9	448	350	499	407.9	0.898	0.858	1.098
702	a	0.436	21	2120	82.9	445	(445)	678	586.0	0.656	0.759	0.759
	b	0.436	21	2885	60.9	520	400	519	494.8	1.002	0.808	1.051

NOTE: In the expression $s\sqrt{f_c'}/3000$, s is bar spacing, center to center, expressed in terms of bar diameters.
*If no P_{yield} was reported, P_{test} is stated in parentheses.

from about 2000 to 5000 psi. These tests were supplemented by 15 similar tests made at the research and development laboratories of the Portland Cement Association in 1954, with concrete strengths of 2000 and 6500 psi, and steel ratio varying from 0.005 to 0.03 (test data in Table 2).

The footings were subjected to a uniform load, and the slabs were loaded through the column stub and supported around their perimeter. These tests make possible an analysis of the effect of the important variables, strength of steel and concrete, size and spacing of bars, position of loading, depth-to-span ratios, and column size, all of which greatly influence the shear strengh.

TESTS REPORTED EARLIER[2]

$\frac{P_{calc}}{P_{flex}}$	$\frac{M_u}{d^2}\sqrt{\frac{d}{l}}$ psi	Reinforcement: No. one way	Size, #	Hooks	Spacing, ϕ	l_b, ϕ	T, kips	u, psi	$\frac{u}{f_c'}$	$s\frac{f_c'}{3000}$	v, psi
84 in., r = 14 in., l = 17.5 in.)											
0.841	226	42	3	S	5.2	50.6	7.28	383	0.109	5.98	316
0.842	227	42	3	S	5.2	50.6	7.28	381	0.104	6.24	316
0.799	248	23	4	H	7.2	38.0	14.9	515	0.155	7.97	293
0.799	250	23	4	H	7.2	38.0	14.9	545	0.143	9.14	313
0.862	206	23	4	H	7.2	38.0	12.2	488	0.128	9.17	303
0.859	205	23	4	H	7.2	38.0	12.2	411	0.110	8.95	254
0.853	211	23	4	S	7.2	38.0	12.5	436	0.113	9.29	269
0.854	210	23	4	S	7.2	38.0	12.5	415	0.116	8.57	254
0.797	250	23	4	S	7.2	38.0	14.9	591	0.144	9.86	338
0.799	250	23	4	S	7.2	38.0	14.9	517	0.132	9.41	297
0.840	216	15	5	H	9.1	30.4	19.8	589	0.162	11.01	290
0.841	215	15	5	H	9.1	30.4	19.8	432	0.132	9.89	211
0.810	236	15	5	H	9.1	30.4	21.9	661	0.206	9.71	312
0.810	239	15	5	H	9.1	30.4	21.9	554	0.143	11.77	263
0.908	181	15	5	H	9.1	30.4	16.5	399	0.138	8.74	212
0.907	182	15	5	H	9.1	30.4	16.5	489	0.150	9.89	261
0.859	203	15	5	S	9.1	30.4	18.6	589	0.151	11.83	296
0.863	205	15	5	S	9.1	30.4	18.6	557	0.129	13.10	284
0.840	216	15	5	S	9.1	30.4	19.8	558	0.155	10.95	276
0.840	216	15	5	S	9.1	30.4	19.8	597	0.160	11.28	295
0.809	238	15	5	S	9.1	30.4	21.9	568	0.149	11.53	268
0.810	237	15	5	S	9.1	30.4	21.9	648	0.185	10.62	304
0.836	223	10	6	H	11.8	25.4	30.5	701	0.192	14.36	274
0.837	224	10	6	H	11.8	25.4	30.5	745	0.194	15.06	293
0.847	213	10	6	H	11.8	25.4	29.3	697	0.208	13.22	276
0.846	214	10	6	H	11.8	25.4	29.3	728	0.194	14.79	288
0.819	235	10	6	H	11.8	25.4	32.1	728	0.187	15.30	278
0.818	235	10	6	H	11.8	25.4	32.1	742	0.185	15.77	285
0.836	221	10	6	S	11.8	25.4	30.5	745	0.243	12.08	290
0.837	224	10	6	S	11.8	25.4	30.5	747	0.196	15.03	292
84 in., r = 14 in., l = 17.5 in.)											
0.852	199	15	5	S	5.67	19	18.3	491	143	10.42	250
0.860	193	15	5	S	5.67	19	18.3	408	208	5.95	204
84 in., r = 14 in., l = 15.75 in.)											
0.819	387	33	4	H	5.0	43.0	12.2	378	0.098	6.30	412
0.817	383	33	4	H	5.0	43.0	12.2	393	0.111	5.79	425
0.864	284	22	4	H	7.6	35.0	16.6	458	0.216	5.30	243
0.953	219	22	4	H	7.6	35.0	12.2	635	0.220	7.21	285

Modes of failure

In spite of the fact that the action of the two-way slab is very complicated, it appears that a simple empirical formula will be suitable for shear design. The slabs and footings under consideration are all slabs reinforced in two directions in which the maximum shear is accompanied by maximum bending moment. This simplifies the problem, because the shear strength is closely related to the bending strength and the applied moment.

The tests showed two different types of failure within which there are variations. One type occurs gradually after the yield stress of the flexural steel has been reached. Excessive cracking of the concrete, due to elongation of the steel, eventually reduces the shear strength until the column punches through.

TABLE 2—SLAB TEST DATA

Test specimen No.	p, percent	d, in.	f_c', psi	f_s, ksi	P_{test}, kips	P_{yield}*, kips	P_{flex}, kips	P_{calc}, kips	$\frac{P_{test}}{P_{flex}}$	$\frac{P_{yield}}{P_{calc}}$	$\frac{P_{test}}{P_{calc}}$
Slabs—Series 1 (a = 72											
A—1a	1.15	4.63	2040	48.2	68	55	81.2	64.3	0.837	0.855	1.058
1b	1.15	4.63	3660	48.2	82	60	89.6	66.1	0.947	0.908	1.241
1c	1.15	4.63	4210	48.2	80	60	87.9	68.0	0.910	0.882	1.176
1d	1.15	4.63	5340	48.2	79	55	89.2	68.8	0.886	0.799	1.148
1e	1.15	4.63	2940	48.2	80	60	85.3	66.5	0.938	0.902	1.203
A—2a	2.47	4.50	1980	46.6	75	(75)	131.0	76.0	0.573	0.989	0.989
2b	2.47	4.50	2830	46.6	90	75	147.2	92.3	0.611	0.813	0.975
2c	2.47	4.50	5430	46.6	105	80	165.1	102.3	0.636	0.782	1.026
7b	2.47	4.50	4050	46.6	115	85	158.3	98.7	0.726	0.861	1.165
A—3a	3.70	4.50	1850	46.6	80	(80)	147.6	72.9	0.542	1.097	1.097
3b	3.70	4.50	3280	46.6	100	(100)	203.9	105.0	0.490	0.952	0.952
3c	3.70	4.50	3850	46.6	120	115	214.6	121.8	0.559	0.944	0.985
3d	3.70	4.50	5010	46.6	123	120	228.8	130.1	0.538	0.922	0.945
Slabs—Series 2 (a = 72											
A—4	1.15	4.63	3790	48.2	90	75	93.9	88.2	0.958	0.850	1.020
5	2.47	4.50	4030	46.6	120	100	171.2	129.6	0.701	0.772	0.926
6†	3.70	4.50									
Slabs—Series 5 (a = 72 in., r = 10 and 14											
A— 9	7.02/1.64	4.50	4330	46.6	100	(100)	147.8	111.5	0.677	0.900	0.900
10	6.28/1.48	4.50	4310	46.6	110	(110)	157.0	152.5	0.701	0.721	0.721
Slabs—Series 7 (a = 72											
A—13	0.55	4.75	3800	42.6	†53	†40	44.1	60.6	1.202	0.660	0.875
Slabs—Series 8 (a = 72											
B— 1	0.50	4.50	2060	47.0	‡40.1	‡30	35.6	42.8	1.126	0.701	0.937
2	0.50	4.50	6900	46.5	‡45.0	‡33	37.0	43.3	1.216	0.762	1.039
4	0.99	4.50	6920	44.0	‡75.0	‡48	67.7	57.4	1.107	0.836	1.307
9	2.00	4.50	6370	49.5	113.5	85	146.2	94.2	0.776	0.902	1.205
11	3.00	4.50	1960	59.3	74.0	(74)	155.9	75.6	0.475	0.979	0.979
14	3.00	4.50	7330	47.2	130.0	100	205.1	119.1	0.634	0.840	1.092

NOTE: In the expression $s\ f_c'/3000$, s is bar spacing, center to center, expressed in terms of bar diameters.
*If no P_{yield} was reported, P_{test} is stated in parentheses.
†Unusual behavior reported, not shown on diagrams.
‡Flexural failure.
**Average of both layers.

Only lightly reinforced slabs reached the load required to develop the full flexural strength calculated by the yield line theory.[4] With heavier reinforcement, the punching failure occurred before the outer bars had reached the yield stress, although the bars near the column had yielded sufficiently to cause the cracking leading to punching. In slabs with the steel uniformly spaced for the full width, this occurred at a load as low as 60 percent of the yield-line-theory load, in which case the flexural steel was only 60 percent efficient. Fig. 1 shows the relation between the ultimate test load and the yield-line-theory load for the test slabs as reported by Elstner and Hognestad.[1]

PREVIOUSLY REPORTED[1]

$\frac{P_{calc}}{P_{flex}}$	$\frac{M_u}{d^2}\sqrt{\frac{d}{l}}$ psi	Reinforcement: No. one way	Size, #	Hooks	Spacing, ϕ	l_b ϕ	T, kips	u, psi	$\frac{u}{f_c'}$	$s\ \frac{f_c'}{3000}$	v, psi
in., r = 10 in., l = 30 in.)											
0.792	183	9	6	S	11.0	32.5	21.2	391	0.192	7.48	251
0.763	192	9	6	S	11.0	32.5	21.2	459	0.126	13.42	303
0.774	201	9	6	S	11.0	32.5	21.2	435	0.103	15.44	295
0.771	205	9	6	S	11.0	32.5	21.2	424	0.079	19.58	291
0.780	194	9	6	S	11.0	32.5	21.2	445	0.151	10.78	294
0.580	255	10	8	S	7.1	24.5	36.8	472	0.238	4.69	287
0.627	338	10	8	S	7.1	24.5	36.8	465	0.164	6.70	345
0.620	389	10	8	S	7.1	24.5	36.8	489	0.090	12.85	401
0.623	371	10	8	S	7.1	24.5	36.8	556	0.137	9.59	439
0.494	239	15	8	S	4.8	24.5	36.8	523	0.282	2.96	306
0.515	403	15	8	S	4.8	24.5	36.8	454	0.138	5.25	383
0.568	489	15	8	S	4.8	24.5	36.8	470	0.122	6.16	458
0.569	531	15	8	S	4.8	24.5	36.8	451	0.090	8.02	472
in., r = 14 in., l = 28 in.)											
0.939	207	9	6	S	11.0	29.8	21.2	411	0.108	13.90	260
0.757	384	10	8	S	7.1	22.4	36.8	485	0.120	9.54	360
in., for A-9 and A-10, respectively; l = 30 in.)											
0.755	436	10	8	S	2.5/10.5	24.5	36.9	431	0.995	3.6/15.2	383
0.971	477	10	8	S	2.5/10.5	22.5	36.9	327	0.758	3.6/15.2	331
in., r = 14 in., l = 28 in.)											
1.374	94	9½**	4.	S	15.3	44.5	8.52	213	0.56	19.38	149
in., r = 10 in., l = 30 in.)											
1.202	85	8½**	4	S	17.8	49.0	9.40	229	0.111	12.22	154
1.170	88	8½**	4	S	17.8	49.0	9.30	251	0.036	40.94	171
0.848	160	10½**	5	S	11.0	39.2	13.60	369	0.053	25.37	286
0.644	348	11	7	S	7.6	28.0	29.7	531	0.083	16.14	435
0.485	253	12	8	S	5.9	24.5	46.9	596	0.304	3.85	283
0.581	486	12	8	S	5.9	24.5	37.3	530	0.072	14.42	496

The ratio of P_{test} to P_{flex} is plotted against the value $M_u/d^2\sqrt{d/l_s}$ to which the shear strength is related as will be shown later. M_u is the moment capacity of the section per in. width, d the depth to steel, and l_s the shear span.

The sudden type of failure before any of the bars have reached yield stress, which occurred in many of the test footings and several of the slabs, can be traced to one of two causes: over-reinforcement in flexure, or bond failure. In the former case, more flexural steel is provided than needed to develop the flexural strength in compression in the concrete, resulting in destruction of the compression zone around the column before the yield stress is reached in the steel. This permits sudden punching. In the latter case, the anchorage of the steel bars in the concrete fails either because of insufficient length of

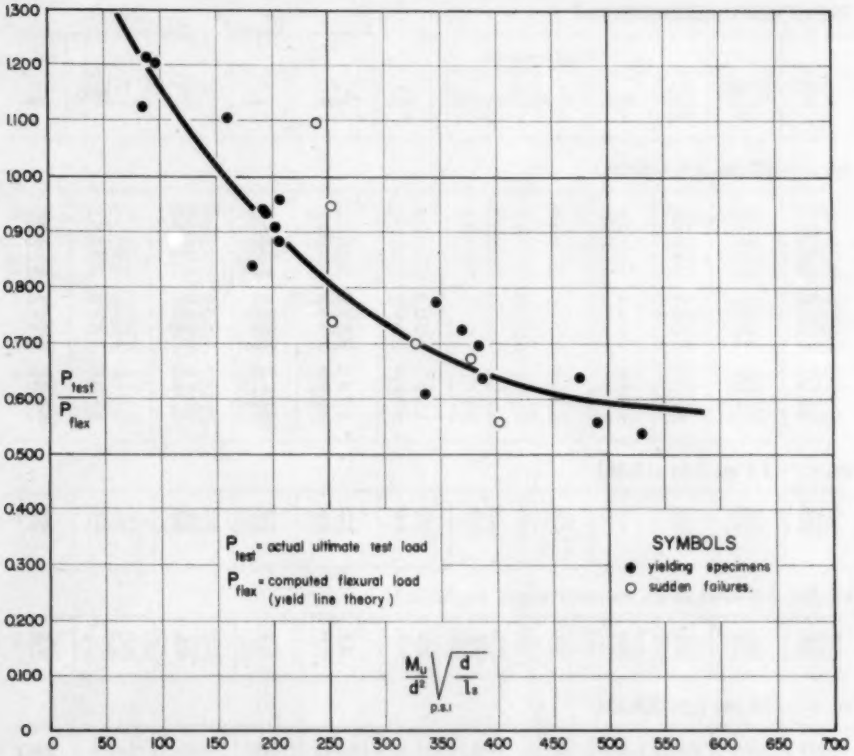


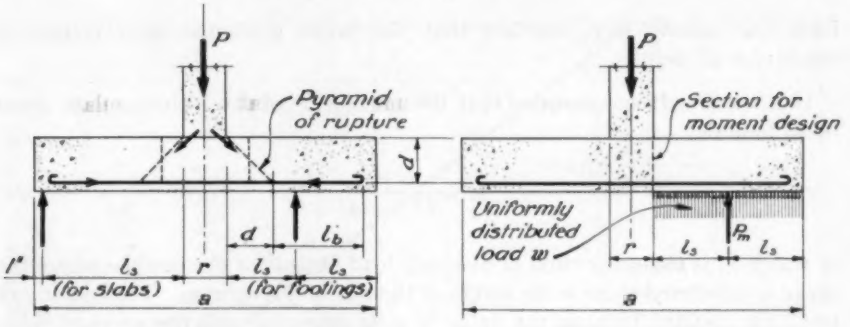
Fig. 1—Efficiency of flexural steel in two-way slabs

embedment or because the bars are too close together. According to the writer's analysis many of the footings failed because of this close spacing, although the average bond stress calculated by the conventional method was low. This finding is consistent with recent reports[5,6] on the effect of bar spacing on bond strength.

Mechanism of failure

Shear failure in the footings and slabs is a local phenomenon depending on the characteristics of the slab in the vicinity of the column. Assuming a square footing with a square column, the failure occurs along pyramidal surfaces sloping out in all directions from the column at an angle of approximately 45 deg (Fig. 2). This will be called the "pyramid of rupture."

The horizontal component of this shearing force must be resisted by the reinforcing steel passing through this pyramid. The limit of this component is the yield strength of the bars, and this can only be effective if the bars are sufficiently anchored in the concrete outside of the pyramid of rupture to develop the full yield strength of the steel. This anchorage length is l_b as shown in Fig. 2. Full development of both flexural and shear strength of the



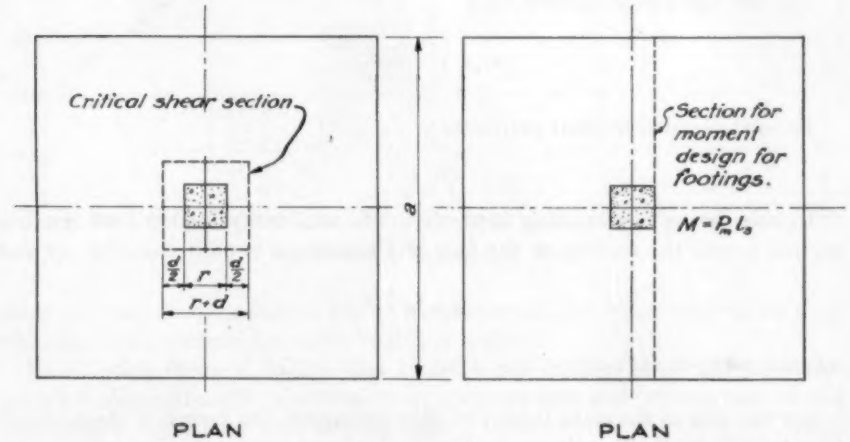


Fig. 2 Fig. 3

footing depends on the ability of these bars to work at yield stress. Calculation of average bond stress across the entire section of the footing by the conventional method is not significant because of the variation in the stress in the bars from center to edge of footing.

As the yield stress develops in the bars passing through the pyramid of rupture, flexural cracks extend up from the steel into the pyramid until they finally precipitate a shear failure; or, if the slab is over-reinforced, the compression zone around the column is destroyed resulting in sudden punching; or, if the steel is not properly anchored in its length l_b, it slips, permitting sudden punching.

Calculations of the shear stress in the test specimens at sections located at the column face, at a distance d from the column face, and at a distance $d/2$

from the column face, indicate that the latter gives the most consistent results for all depths.

It is therefore recommended that the calculation of the ultimate unit shear be as follows:

$$v_u = \frac{k_1 P_{ult}}{4d\,(r + d)}$$

in which k_1 is the proportion of ultimate load outside of the surface where the shear is calculated and r is the length of the side of the column. The customary term j is omitted because the value of v_u is empirical and the average value for the full depth is as good as any other.

For footings under uniform load

$$k_1 = 1 - \frac{(r + d)^2}{a^2}$$

For slabs loaded at their perimeter

$$k_1 = 1$$

In calculating the bending moment in the uniformly loaded footings, the section across the footing at the face of the column is used (see Fig. 3,) and

$$M = P\,\frac{2\,l_s^2}{a}$$

where $l_s = (a - r)/4$.

For the case of the slabs loaded at their perimeter, the term l_s is the distance from the face of the column to the load at the corresponding edge. The moment is calculated by the yield-line-theory equations given by Elstner and Hognestad.*

Ultimate shear strength

The ultimate shear strength is closely associated with the ultimate resisting moment per unit width of slab near the column, which is principally a function of the value of pf_y for under-reinforced slabs or of f_c' for over-reinforced slabs. The shear strength is not very sensitive to variation in the concrete strength in normal cases which have less than balanced flexural reinforcement except when the concrete strength is too low to provide proper anchorage for the steel. For this reason, the unit shear strength cannot properly be expressed as a simple percentage of the concrete cylinder strength as assumed in the present conventional method.

The shear strength is also affected by the relation between the total horizontal compressive stress and the shear stress, which depends on the relation of the depth of the slab to the position of the load, d/l_s. Barring bond failure,

*See p. 55 of Reference 1.

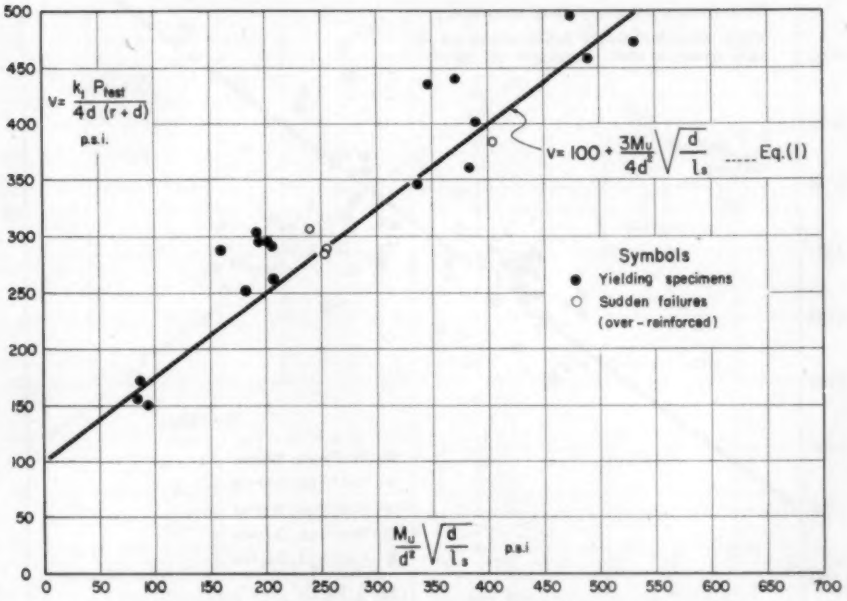

v= k₁ P_test / 4d (r + d) p.s.i.
v= 100 + 3M_u/4d² √(d/l_s) ..... Eq.(1)
Symbols
Yielding specimens
Sudden failures (over-reinforced)
M_u/d² √(d/l_s) p.s.i
0 50 100 150 200 250 300 350 400 450 500 550 600 650 700
0 50 100 150 200 250 300 350 400 450 500

Fig. 4—Edge-loaded slab tests

relatively deeper slabs show a higher shear strength for equal steel ratios than the thin slabs because the value of d/l_s is higher.

Eliminating cases of failure due to improper anchorage of the bars, the ultimate shear strength calculated from both the slab and footing tests shows an excellent agreement with the equation

$$v = 100 + 0.75 \frac{M_u}{d^2} \sqrt{\frac{d}{l_s}} \quad \text{.....(1)}$$

in which l_s is the distance from face of column to load position and M_u is the ultimate resisting moment per in. width of slab within the base of the pyramid of rupture.

$$M_u = pd^2 f_y \left[1 - \frac{pf_y}{1.7 f_c'} \right], \text{ when under-reinforced}$$

$$M_u = \frac{d^2 f_c'}{3}, \text{ when over-reinforced}$$

Fig. 4 shows the results of the slab tests for comparison with Eq. (1) (omitting Slab II A-6 because it was not consistent with the other tests and had bars too closely spaced.) Four other over-reinforced slabs, A-2a, A-3a, A-3b, and B-11 behaved well in spite of close bar spacing. The fact that they were not called upon to develop yield stresses in the steel no doubt made the anchorage more effective than it would otherwise have been. It should be noted that all

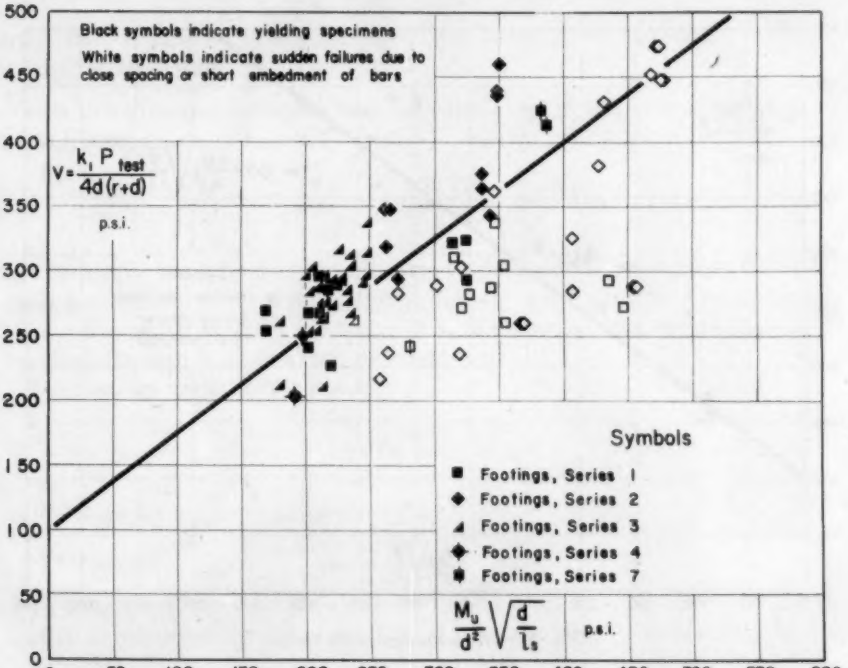


Fig. 5—Footing tests

slabs which failed at or near the yield load (A-2a, A-3a, A-3b, A-3c, A-3d, A-6 and B-11) were over-reinforced or had bars spaced very close.

The results of footing tests reported by Richart and Hognestad[2,3] are plotted in Fig. 5 with those footings identified which failed suddenly because of steel anchorage failure due to close spacing or short embedment.

Fig. 6 shows all slab and footing tests excepting the footings which failed suddenly.

Additional studies have been made to determine the effects of depth and concrete strength on the plotting. No trend was discovered which would indicate that Eq. (1) is not satisfactory.

To test the location of the critical shear section, Fig. 7 and 8 were plotted with $v = (k_3 P_{test})/[4d (r + 2d)]$ and $k_2 (P_{test})/(4 dr)$ respectively. In the former, the results of footing tests lie generally below the slabs which are thinner than the footings, while in the latter, they are generally above the slabs. Use of the section $d/2$ from the column gives better correlation for all depths.

Anchorage of reinforcing steel

According to the writer's analysis, most cases of failure which occurred substantially below the shear value given by Eq. (1) can be traced to bond failure caused by close spacing or short embedment of the bars outside of the

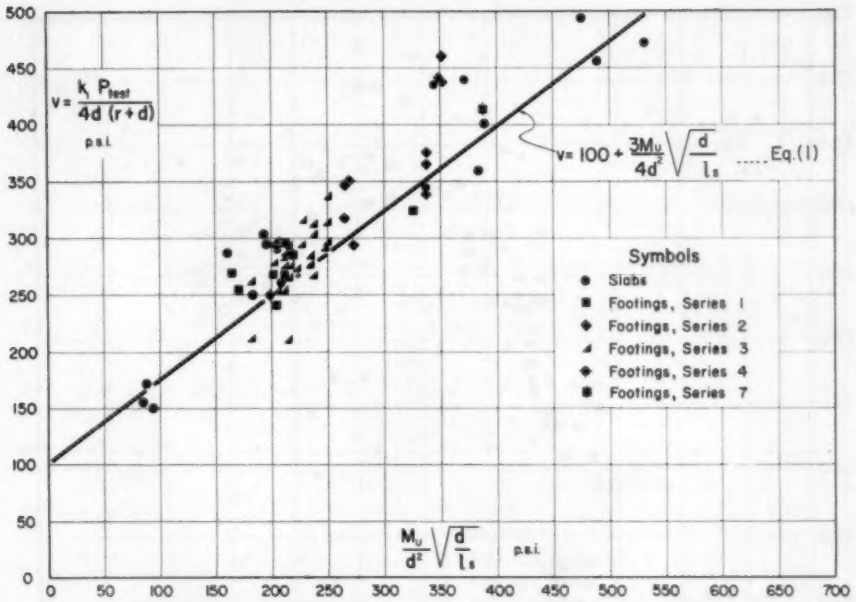

$v = \frac{k_1 P_{test}}{4d\,(r+d)}$ p.s.i.
$v = 100 + \frac{3M_u}{4d^2}\sqrt{\frac{d}{l_s}}$ .....Eq.(1)
Symbols
Slabs
Footings, Series 1
Footings, Series 2
Footings, Series 3
Footings, Series 4
Footings, Series 7
$\frac{M_u}{d^2}\sqrt{\frac{d}{l_s}}$ p.s.i.
0 50 100 150 200 250 300 350 400 450 500 550 600 650 700

Fig. 6—Slab and footing tests, except steel anchorage failures

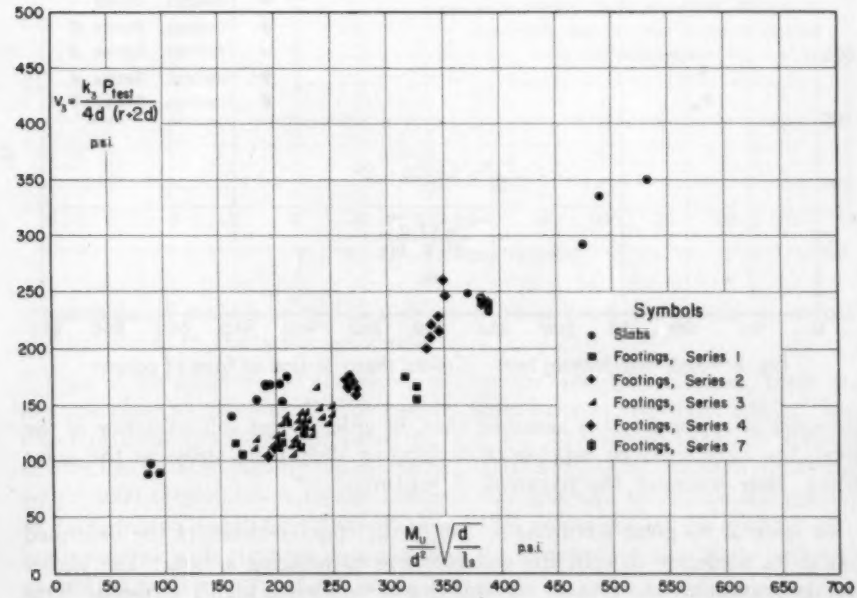


Fig. 7—Slab and footing tests, yielding specimens. Critical shear at distance d from face of column

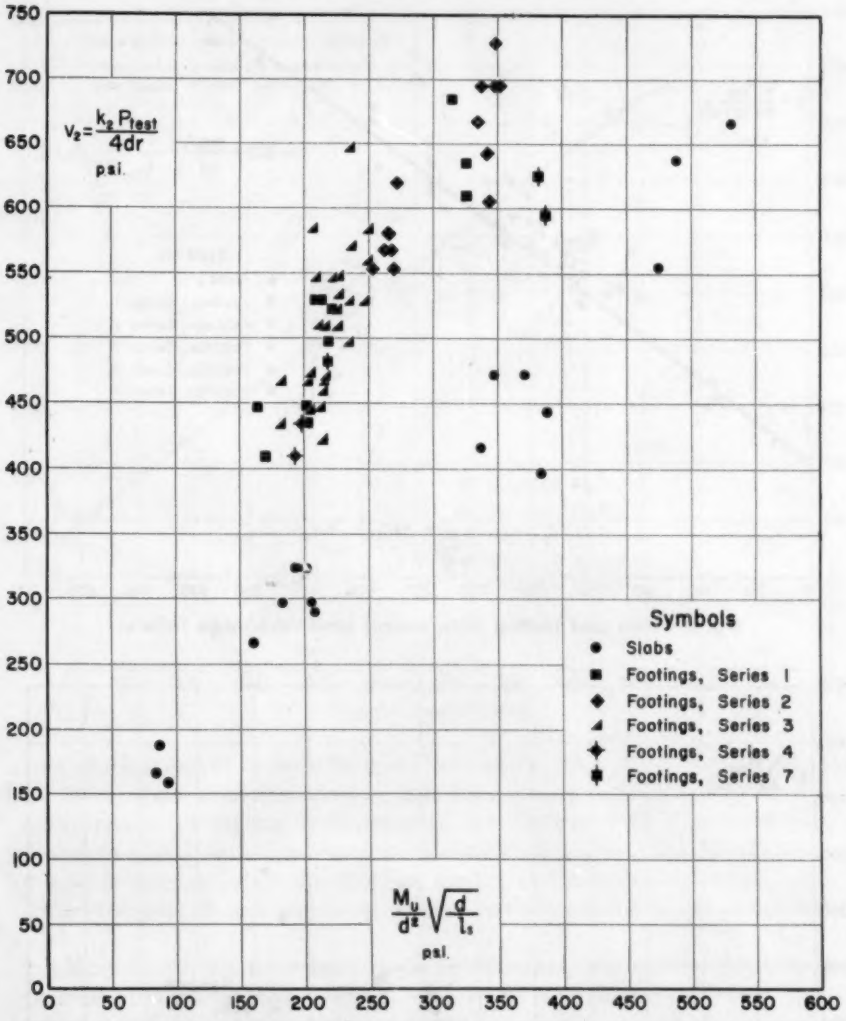


Fig. 8—Slab and footing tests. Critical shear section at face of column

pyramid of rupture. It is assumed that, in order to get full efficiency of the steel, the bars must be capable of developing their yield stress at the point where they intersect the pyramid of rupture.

In spite of its great advantages, one of the characteristics of the deformed bar is its tendency to split the concrete due to wedging action. The higher the deformations, the greater the splitting action when highly stressed. This has been noted by a number of investigators of bond strength in beams. In the case of footings and slabs with two layers of steel, this action is greatly

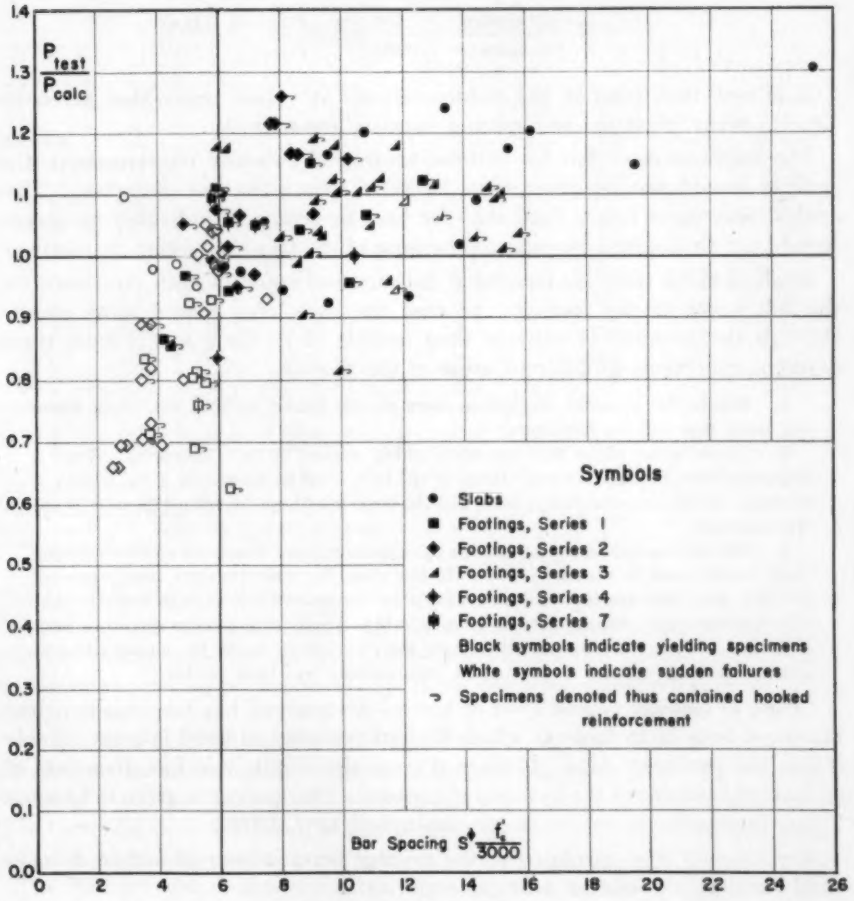


Fig. 9—Effect of bar spacing

magnified and there is no doubt additional loss of bond due to lack of embedment adjacent to the bar-to-bar contacts.

Critical bar spacing—In the series of footings tested by Richart, there are in some cases as many as 60 bars equally spaced across the 84-in. width. The effect of close spacing is indicated by Fig. 9. The center-to-center spacing of bars has been divided by the diameter of the bar and this has been multiplied by $f_c'/3000$ to give effect to the concrete strength and to reduce the spacing in diameters to a standard 3000-psi case. It is assumed that the critical spacing is smaller with stronger concrete, and the assumption of a linear variation should be satisfactory for the range involved.

This value is plotted in Fig. 9 against the value of ultimate test load divided by the ultimate load given by Eq. (1), that is

$$\frac{\text{bar spacing}}{\text{bar diameter}} \times \frac{f_c'}{3000} \text{ versus } \frac{P_{test}}{P_{calc}}$$

It is seen that most of the sudden failures at values below that given by Eq. (1) occur when the bar spacing index is less than 6.

The implication is that for uniform spacing of two-way reinforcement, the spacing should not be closer than $18{,}000/f_c'$ times the bar diameter. This critical spacing is larger than that for bars in beams as indicated by recent tests[5,6] but that should be expected because of the two-way action in footings.

In all of these tests the spacing is uniform and equal in both directions for the full width of the footing. In case the bars were spaced more closely through the pyramid of rupture than outside of it, there would exist three different conditions in different areas of the footing:

1. Within the pyramid of rupture, bars closely spaced in both directions where the bond may not be important.
2. Corner areas where bars are more widely spaced in both directions. Bond is important here because the yield stress in the bars must be developed if the footing is to carry its full yield-line-theory load, but the bond would not be critical because of the wide spacing.
3. The intermediate areas outside the pyramid of rupture where the main reinforcing must be anchored to prevent failure. In this area, the main bars are closely spaced, but the cross bars are farther apart. Tests on beams without stirrups indicate that this condition may not be quite as critical as if the bars were equally spaced in both directions, but in the absence of definite information it would be conservative to assume that the spacing should not be substantially less than $18{,}000/f_c'$.

Length of embedment and effect of hooks—An analysis has been made of the results of tests of 15 footings which Richart reported as bond failures. Table 3 lists the pertinent data. Column 6 gives the length l_b in bar diameters of embedment outside of the pyramid of rupture. The spacing is given in Column 7 (bar diameters, center to center, multiplied by $f_c'/3000$).

The value of u is calculated as the average bond stress required to develop yield stress in the steel in a length equal to l_b.

It is assumed that in the ideal case when P_{test} equals P_{calc}, the stress in the steel will be equal to f_y. In Column 8, u has been multiplied by P_{test}/P_{calc} to indicate what effective bond stress would be developed in the test.

Column 9 is the effective bond stress divided by f_c'. The tests indicate that a value of $0.2f_c'$ could be assumed as the ultimate bond strength up to a value of $f_c' = 3500$, putting a top limit of 700 psi on u. The adjusted bar spacing $s\ f_c'/3000$ varied from 12.1 to 25.9.

Column 10 shows that the agreement of the tests with this assumption is very satisfactory. This would require a minimum embedment of 14.3 diameters for $f_y = 40{,}000$ psi and 21.5 diameters for f_y 60,000 for new deformed bars. It is suggested that the ultimate value of u for plain bars be assumed as ½ that for deformed bars.

This method of arriving at the bond strength from the tests may seem arbitrary, but actually it should be satisfactory. The average value of bond

TABLE 3—ANALYSIS OF 15 FOOTING FAILURES

1	2	3	4	5	6	7	8	9	10
Test specimen, No.	f_c', psi	f_y, ksi	Bars: Type	Bars: Size, #	l_b, in bar diameters	$s\frac{f_c'}{3000}$, bar diameters	$u\frac{P_{test}}{P_{calc}}$, psi	$\frac{u}{f_c'}\cdot\frac{P_{test}}{P_{calc}}$	$\frac{u}{U_u}\cdot\frac{P_{test}}{P_{calc}}$
321 a	3650	69.2	NDR – H	6	25.4	14.4	701	0.191	1.00
b	3830	69.2	NDR – H	6	25.4	15.1	745	0.193	1.06
323 a	3360	66.5	ODR – H	6	25.4	13.2	697	0.206	1.03
b	3760	66.5	ODR – H	6	25.4	14.8	728	0.194	1.04
324 a	3890	73.0	PR – H	6	25.4	15.3	728	0.187	1.04
b	4010	73.0	PR – H	6	25.4	15.8	742	0.184	1.06
325 b	3520	52.7	NDB – S	6	25.4	13.8	617	0.175	0.88*
326 a	3070	69.2	NDR – S	6	25.4	12.1	745	0.242	1.21
b	3820	69.2	NDR – S	6	25.4	15.0	747	0.195	1.07
328 a	3080	66.5	ODR – S	6	25.4	12.1	649	0.211	1.06
b	3610	66.5	ODR – S	6	25.4	14.2	648	0.179	0.93
330 a	3500	59.7	NDB – S	8	19.0	18.4	747	0.214	1.07
b	3630	59.7	NDB – S	8	19.0	19.1	747	0.206	1.07
331 a	3705	59.0	NDB – S	10	15.2	25.9	990	0.267	1.41
b	2790	59.0	NDB – S	10	15.2	19.5	817	0.293	1.41

ODR:Old deformed rail
NDR:New deformed rail
NDB:New deformed billet
U_u:Ultimate bond for design ($U_u = 0.2\,f_c' < 700$ psi)
*Probable tensile failure

PR:Plain rail
H:Hooked bar
S:Straight bar

over the full length l_b decreases as l_b increases. All footings except the 15 in Table 3 had greater values of l_b and it is believed they failed primarily in tension (or diagonal shear induced by tensile yielding) or because of splitting due to close spacing of the bars. In those cases the average bond stress is not significant.

In case of a long embedment, the bond failure is progressive, so in general the shorter the embedment, the greater the unit bond strength. This is illustrated by Footings 331a and 331b with a 15-in. l_b and much higher value of u. It is therefore believed that the use of the value of u indicated by the cases with l_b equal to 25 as shown in Table 3 is conservative.

The Richart tests indicate conclusively that, with deformed bars, hooks at the ends of the bars added nothing to the bond strength. This is shown by Table 3 to be true even when l_b is as short as 25 diameters. Failure of the footing is normally due to stretching of the bars without any slipping at their ends. It makes no difference whether they are anchored or not if l_b is adequate.

In cases where l_b is too short to develop the yield strength of the bars, it is suggested that hooks may be used to make up the deficiency but not to be counted on to develop more than 20,000 psi of the ultimate stress in the bars.

Effect of concentration of steel at columns

The Building Code (ACI 318-56) provides that a 20 percent greater working stress in shear can be used around columns in flat slabs when at least 50 percent of the total negative reinforcement required for bending in the column strip passes through the periphery at distance d beyond the edges of the column or column capital. No record can be found of the reason for this provision nor of any tests to support it. For this reason it is interesting to examine the

behavior of two slabs (A-9 and A-10) reported by Elstner and Hognestad which had half of the steel concentrated under the columns.

These slabs were stiffer and carried a higher load before yielding than their counterparts, Slabs A-7b and A-5 which had the same total amount of steel uniformly spaced.

It is important to recognize that both A-9 and A-10 were heavily over-reinforced at the columns, so that the steel could not be fully effective. The actual p for A-9 was 0.0702 whereas the balanced ratio is

$$p = 0.456 \frac{f_c'}{f_y} = 0.042$$

In addition to this, the bars were 1 in. in diameter, spaced 2½ in. center to center. Judging from the behavior of footings with closely spaced bars, this could have caused a bond failure before the full moment capacity was reached, accounting for the apparently low shear strength.

Note that strength of Slab A-9 is the same as that of A-36 with about the same concrete strength but over-reinforced for its full width.

Obviously more tests are needed but nothing in these tests indicates that the shear strength does not depend on the effective amount of reinforcing passing through the pyramid of rupture. If this is true, it will be possible to balance shear strength with flexural strength of a slab or footing by concentrating steel near the column. There appears to be no other method of achieving this balance except in the case of very light reinforcement (Fig. 1).

That this method of concentrating the steel under the column has appeared logical to designers in the past is indicated by the recommendations for reinforced concrete footing design presented in the 1909 edition of Taylor and Thompson's *Treatise on Concrete, Plain and Reinforced.* They state: "Find the area of steel required for the whole circumference of the inner circle of the plate . . . and place it in two directions, at right angles, distributing it over an area slightly larger than the base of the column. Double the spacing of rods outside of the column, as the bending moment decreases very rapidly . . . "

Shear reinforcement

Tests of a number of slabs with various types of shear reinforcement indicated that it is possible to increase the shear resistance by such means. However, it appears probable that when the shear strength as indicated by Eq. (1) is properly recognized, it will be found that shear reinforcement is unnecessary and uneconomical. Shear reinforcement of slabs is expensive and inefficient because of the poor anchorage and it is believed that increased shear capacity can best be provided by using additional flexural steel through the pyramid of rupture which will at the same time improve the flexural strength and stiffness and reduce cracking.

SUGGESTIONS FOR ULTIMATE STRENGTH DESIGN OF FOOTINGS AND FLAT SLABS

This analysis of tests leads to two principal conclusions:

First, the present method of shear design is inadequate because the allowable shear is not based on the proper factors. The results are inconsistent, being in general, too conservative for cases with heavy flexural reinforcement and relatively unsafe with light reinforcement. Too much weight is given to f_c' and not enough to the steel ratio and shear span.

Second, as indicated by Fig. 1, except in cases of unusually light flexural reinforcement, it is not possible to provide a design with uniformly spaced bars which will have equal flexural and shear capacity. This can be accomplished only by using closer spacing of the rods through the pyramid of rupture. It appears that the correct procedure would be to first calculate the total amount of reinforcement needed for flexure, placing enough steel through the pyramid of rupture to provide consistent shear strength. The balance of the total steel should then be distributed outside of the pyramid of rupture.

A few tests were made on rectangular footings and on slabs with eccentrically loaded column stubs. These indicate no particular complications, and it is suggested that such cases can be designed satisfactorily by designing the bands in different directions separately. The rectangular footing presents no problem because the static moment and shear are to be calculated in each direction as in the case of the square footing with proper consideration for the geometry.

Design of footings

In designing footings by the ultimate strength method, a load factor of 2 appears appropriate because 2 has been recommended for the columns supported on the footings, and also because, even at ½ the ultimate load, considerable cracking can be expected in the concrete at the bottom surface making a factor as low as 1.8 undesirable. The sum of the dead and live loads can then be multiplied by 2 and the footing designed for that ultimate strength.

The total amount of steel in each direction should be calculated from the total static moment at the section across the footing at the face of the column. The tests do not show any justification for any reduction in this moment in figuring the reinforcement as is presently allowed in ACI 318-56.

After the total steel requirement is determined, the amount required through the pyramid of rupture for shear is calculated and the balance evenly distributed outside of the pyramid of rupture.

A check is then made on the spacing and anchorage of the rods to see that the spacing is not less than $18{,}000/f_c'$ diameters, center to center, and that the bond stress to develop the specified yield strength of the rods in the length l_b is not greater than $0.2\,f_c'$ or 700 psi.

Because of the two-way reinforcement, the economical amount of reinforcing each way in a footing may be as low as ½ to ¾ percent. Beyond a certain

point, deepening the footing will cost more in added concrete and excavation than the saving in reinforcing steel.

Designing a footing by this new method will require trial of different proportions to determine the most economical design until more experience has been gained, but the method appears fully as simple as the present one.

Design of flat slabs

For the ultimate strength design of flat slabs in flexure and shear, it is suggested that a load factor of 1.8 may be used. The steel requirements for flexure and shear may be adjusted as in the case of footings to secure a balanced design.

In applying Eq. (1), it is to be assumed that the slab is a square or rectangular slab extending out to the theoretical lines of inflection and that the load is applied around that perimeter. The value of l_s may be assumed to be the distance from the face of the column or cap to the line of inflection. This is probably a conservative assumption because some of the load is distributed inside the lines of inflection and the value of l_s would thereby be reduced.

As in the case of footings, the spacing and the anchorage of the bars should be checked. The value of l_b will be the distance from the pyramid of rupture to the line of deflection, where presumably the stress in the bars will be zero.

The economical steel ratio is higher for floor slabs than for footings because the slabs support their own weight and because there is an architectural premium on a not-too-great thickness. This will make it desirable to take advantage of the much higher shear strength provided by the flexural steel, even in some cases adding bars through the zone of rupture in order to control the shear strength.

In the case of eccentric loading, the distribution of load around the perimeter following the lines of contraflexure can be estimated according to the straight line distribution theory and the shears in the different bands calculated accordingly.

BEAMS AND FRAME MEMBERS

This new method of estimating the shear strength of simple beams under concentrated or uniformly distributed loading and of frame members under concentrated load with axial compression or tension is based largely on the analysis of two test programs; it is realized that a study of other tests must be made to determine whether or not the equations can be improved.

It is suggested that the shear strength depends on the ultimate flexural capacity of the member and on the ratio of depth to shear span. It is therefore largely a function of the strength of the reinforcement and the proportions of the beam, and not primarily dependent on f_c' as assumed in the present conventional method.

It is further suggested that the unit shear should be considered as V/bd instead of V/bjd as in the straight line method. The value j is not used with the ultimate strength method and has no real significance. In this section on

beams and frame members, however, v is taken as V/bjd for convenience in comparison with the reported test results and code values, but the equations involving v can be easily adjusted by multiplying the numerical coefficients by 0.875.

Although the maximum load was used in calculating the shear strength of slabs and footings, it is recommended that the diagonal cracking load be used in calculating the ultimate shear strength for beams without shear reinforcement. In order to avoid confusion it must be remembered that in this section on beams and frame members v_u is calculated from the diagonal cracking load and not from the maximum load carried by the beam. This will be considered the ultimate shear strength in applying the ultimate strength method. This is justified because the integrity of a beam without shear reinforcement is lost when diagonal cracking occurs, and repeated loading may cause failure.

This is not necessarily true in the case of a two-way slab or footing where some local flexural yielding may occur without danger of failure. In the latter case, the diagonal cracking load cannot be determined by test because the transition from vertical to diagonal cracks cannot be observed.

The ultimate strength of the compression side of a beam is affected by the obliquity of the resultant compression force which is a function of the shear span l_s and the distance c from the axis of the reinforcement to the center of concrete compression. From the plastic theory[7] this value of c is

$$c = d\left(1 - \frac{pf_y}{1.7\,f_c'}\right)$$

The limiting value of c is $0.732d$ for beams with balanced reinforcement and no axial thrust; and it approaches unity for light reinforcement.

The use of c instead of d complicates the calculations; and although the theory agrees with the tests somewhat better if c is used it is thought that d can be used with sufficient accuracy for beams under flexure alone and also in the case of flexure and direct load when the eccentricity is not too small.

Simple beams under concentrated loading

Five years ago a series of tests on stub beams and knee frames was recommended by the joint ACI-ASCE committee on shear and diagonal tension for the purpose of determining what proportion of the ultimate flexural capacity of simple beams and frame members can be developed under concentrated load without shear reinforcement.

These tests have been carried out at the University of Illinois under the sponsorship of the Reinforced Concrete Research Council and have been reported by Morrow and Viest.[8]

Stub beams with the central load applied through short integral column stubs were used so that they could be compared with knee frames to determine the effects of the direct load on the shear strength. The concrete strength covered a range of 1470 to 7020 psi, the steel ratio from 0.0034 to 0.0385 and

steel strength from 48,000 to 68,000 psi. Shear spans varied from 14 to 113 in. with d from 14¼ to 16 in.

The unit shear at diagonal tension cracking load for the stub beams was plotted by the writer against the value

$$\frac{M_u}{d^2}\sqrt{\frac{d}{l_s}}$$

in which l_s is the shear span, and M_u is the ultimate moment capacity per in. width of beam.

$$\frac{M_u}{d^2} = \frac{f_c'}{3} \text{ for over-reinforced beams}$$

$$\frac{M_u}{d^2} = pf_y\left(1 - \frac{pf_y}{1.7\,f_c'}\right) \text{ for under-reinforced beams}$$

Balanced reinforcement is given by

$$p_b = 0.456\,\frac{f_c'}{f_y}$$

The test data for beams under concentrated load are summarized in Table 4 and the results are shown in Fig. 10. The scatter which is largely on the upper side of the line

$$v_u = 50 + 0.26\,\frac{M_u}{d^2}\sqrt{\frac{d}{l_s}} \qquad (2)$$

appears to be due to the influence of the bar spacing and its effect on bond strength. The beams were 12 in. wide and were reinforced with from three to seven bars. For instance B28A6 has two #8 and five #9 bars so that the spacing is extremely close. The shear values substantially above the line are for beams with three, four or five bars. Table 4 gives the relations between v_{test}/v_{calc} and the number of bars.

The correlation is so good that it may be better to use the conservative value of the ultimate shear given by Eq. (2), rather than attempt to take advantage of the strength gained by wider spacing. The latter would make practical design very difficult and would require considerable additional research to determine the exact relation.

Eq. (2) gives the shear at the first diagonal cracking load which for design purposes should probably be considered as the ultimate strength of a beam without shear reinforcement.

The relation between v_u and $(M_u/d^2)\sqrt{c/l_s}$ is also given in Fig. 10. This indicates that the use of c instead of d may give slightly better results especially for the smaller values of l_s. The ultimate shear v_u is

$$v_u = 50 + 0.3\frac{M_u}{d^2}\sqrt{\frac{c}{l_s}} \quad \text{.....(2a)}$$

By means of Eq. (2) it is possible to derive an equation for the length of a beam without shear reinforcement which will have equal flexural and shear capacity under concentrated load. Equating the ultimate moment capacity to the product of the ultimate shear strength multiplied by the shear span

$$M_u = v_u jdl_s = 50jdl_s + 0.26jdl_s\frac{M_u}{d^2}\sqrt{\frac{d}{l_s}}$$

solving l_s/d, with $j = 0.875$

$$\sqrt{\frac{l_s}{d}} = \sqrt{\left(\frac{M_u}{385d^2}\right)^2 + \frac{M_u}{43.7d^2}} - \frac{M_u}{385d^2} \quad \text{.....(3)}$$

For balanced reinforcement when $M_u/d^2 = f_c'/3$

$$\sqrt{\frac{l_s}{d}} = \sqrt{\left(\frac{f_c'}{1155}\right)^2 + \frac{f_c'}{131}} - \frac{f_c'}{1155} \quad \text{.....(4)}$$

In this case, the length of simple span under concentrated central load is $L = 2\ l_s$. The following proportions result from Eq. (3):

$$\text{When } \frac{M_u}{d^2} = 500,\ L/d = 10.8$$

$$\frac{M_u}{d^2} = 1000,\ L/d = 15.7$$

$$\frac{M_u}{d^2} = 2000,\ L/d = 21.8$$

and from Eq. (4):

$$\begin{aligned} \text{When } f_c' &= 2000,\ L/d = 12.9,\ v_u = 118 \text{ psi or } 0.059f_c' \\ f_c' &= 3000,\ L/d = 15.7,\ v_u = 143 \text{ psi or } 0.048f_c' \\ f_c' &= 4000,\ L/d = 18.5,\ v_u = 164 \text{ psi or } 0.041f_c' \end{aligned}$$

Longer beams with greater L/d need no shear reinforcement. Shorter beams need shear reinforcement for at least part of their length. As shown by Eq. (3) the smaller the steel ratio, the shorter the critical length.

The efficiency of a simple beam under concentrated load which is shorter than the critical length can be obtained by dividing the ultimate shear moment by the flexural capacity as follows:

TABLE 4—STUB BEAMS

Specimen		p, %	f_s, ksi	f'_c, psi	b, in.	d, in.	l_s, in.	Reinforcing bars		$\frac{M_u}{d^2}\sqrt{\frac{d}{l_s}}$ psi	v_u, psi	v_{calc}, psi	$\frac{v_u}{v_{calc}}$
B14	B2	1.85	67.6	2120	12.00	14.50	14.00	2 #6	4 #7	832	230	266	0.865
	E2	0.57	67.5	1840	12.00	14.75	14.00	2 #4	2 #5	346	194	140	1.386
	A4	2.50	61.7	3270	12.00	14.25	14.00	2 #7	4 #8	1124	334	342	0.977
	B4	1.85	58.6	3820	12.00	14.50	14.00	2 #6	4 #7	920	328	289	1.135
	E4	1.24	61.2	4190	12.00	14.50	14.00	5 #6		691	345	230	1.500
	A6	3.83	65.0	6590	12.00	14.00	14.00	2 #8	5 #9	1936	425	553	0.769
	B6	1.85	65.9	6780	12.00	14.50	14.00	2 #6	4 #7	1111	328	339	0.968
B21	B2	1.86	63.4	2010	12.00	14.44	21.00	2 #6	4 #7	642	231	217	1.065
	E2	0.57	67.2	1640	12.00	14.75	21.00	2 #4	2 #5	277	145	122	1.189
	A4	2.46	58.7	4320	12.00	14.50	21.00	2 #7	4 #8	966	296	301	0.983
	B4	1.85	61.3	3930	12.00	14.50	21.00	2 #6	4 #7	784	263	254	1.035
	E4	1.24	62.4	3510	12.00	14.38	21.00	5 #6		558	295	195	1.513
	E4R	1.24	60.4	4630	12.00	14.50	21.00	5 #6		564	263	197	1.335
	F4	1.17	66.2	4560	12.00	14.56	21.00	2 #7	1 #8	582	294	201	1.463
	G4	0.58	67.8	4580	12.00	14.69	21.00	2 #4	2 #5	312	243	131	1.855
	A6	3.86	64.9	6570	12.00	14.00	21.00	2 #8	5 #9	1578	337	460	0.733
	B6	1.82	63.4	6600	12.00	14.75	21.00	2 #6	4 #7	869	323	276	1.170
B28	B2	1.88	68.3	2130	12.00	14.25	28.00	2 #6	4 #7	592	184	204	0.902
	E2	0.57	67.2	1990	12.00	14.63	28.00	2 #4	2 #5	245	129	114	1.132
	A4	2.46	48.2	3990	12.00	14.50	28.00	2 #7	4 #8	703	223	233	0.957
	B4	1.85	64.0	4690	12.00	14.50	28.00	2 #6	4 #7	726	197	239	0.824
	E4	1.24	62.2	4800	12.00	14.50	28.00	5 #6		503	181	181	1.000
	A6	3.83	66.0	6840	12.13	13.88	28.00	2 #8	5 #9	1396	322	413	0.780
	B6	1.85	65.5	6350	12.00	14.50	28.00	2 #6	4 #7	773	263	251	1.048
B40	B4	1.85	54.8	5040	12.00	14.50	40.00	2 #6	4 #7	539	228	190	1.200
B56	B2	1.85	68.3	2130	12.00	14.50	56.00	2 #6	4 #7	419	148	159	0.931
	E2	0.58	67.0	2130	12.00	14.50	56.00	2 #4	2 #5	176	99	96	1.030
	A4	2.41	47.8	3620	12.00	14.75	56.00	2 #7	4 #8	482	200	175	1.143
	B4	1.85	63.9	3950	12.00	14.50	56.00	2 #6	4 #7	497	181	179	1.011
	E4	1.24	62.2	4120	12.00	14.50	56.00	5 #6		350	161	141	1.142
	A6	3.79	63.6	5780	12.13	14.00	56.00	2 #8	5 #9	910	269	287	0.937
	B6	1.83	67.6	6630	12.00	14.63	56.00	2 #6	4 #7	566	202	197	1.025
B70	B2	1.86	67.0	2370	12.00	14.38	70.00	2 #6	4 #7	391	132	152	0.868
	A4	2.46	63.2	3950	12.00	14.50	70.00	2 #7	4 #8	544	195	191	1.021
	A6	3.83	63.1	6520	12.00	14.00	70.00	2 #8	5 #9	848	272	270	1.007
B84	B4	1.88	67.4	3950	12.00	14.31	84.00	2 #6	4 #7	425	166	161	1.031
B113	B4	1.86	68.0	4730	12.00	14.38	113.00	2 #6	4 #7	381	155	149	1.040
	B4R	1.86	50.0	4160	12.00	14.50	113.00	2 #6	4 #7	290	—	125	

$$\frac{M_v}{M_u} = \frac{50jdl_s}{M_u} + \frac{0.26jdl_sM_u}{M_u d^2}\sqrt{\frac{d}{l_s}}$$

which reduces to

$$\frac{M_v}{M_u} = 43.7\frac{d^2}{M_u}\frac{l_s}{d} + 0.2275\sqrt{\frac{l_s}{d}} \quad \text{.....(5)}$$

Applied to a beam with a length of $10d$ and $M_u/d^2 = 1000$ this gives

$$\frac{M_v}{M_u} = \frac{43.7 \times 5}{1000} + 0.2275\sqrt{5} = 0.73$$

and shows that it will fail in shear when 73 percent of the moment capacity is developed. However, this deficiency in shear strength does not exist over the entire span. The shear strength increases from the center of the span toward the support as the shear span decreases, and Eq. (5) can be used to

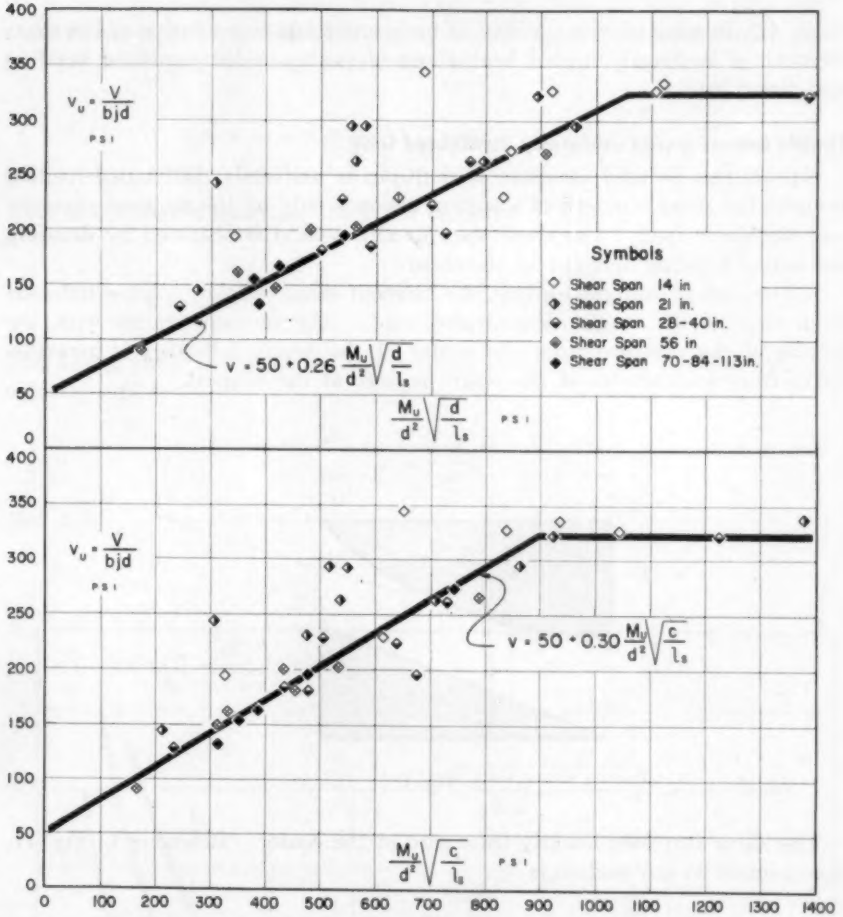

$v_u = \frac{V}{bjd}$ P.S.I.
Symbols
Shear Span 14 in.
Shear Span 21 in.
Shear Span 28-40 in.
Shear Span 56 in.
Shear Span 70-84-113in.
$v = 50 + 0.26 \frac{M_u}{d^2}\sqrt{\frac{d}{l_s}}$
$\frac{M_u}{d^2}\sqrt{\frac{d}{l_s}}$ P.S.I.
$v_u = \frac{V}{bjd}$ P.S.I.
$v = 50 + 0.30 \frac{M_u}{d^2}\sqrt{\frac{c}{l_s}}$
$\frac{M_u}{d^2}\sqrt{\frac{c}{l_s}}$ P.S.I.
0 50 100 150 200 250 300 350 400
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400

Fig. 10—Stub beams. Unit shear at diagonal tension cracking

determine what portion of the beam is deficient in shear strength. It is possible that Eq. (5) may also be used to determine how much shear reinforcement is needed because it appears probable that the amount needed is a function of the relation between shear strength and flexural strength of the beam without shear reinforcement. This problem obviously needs further study.

In all cases it has been assumed that the flexural steel is carried full length of the beam. If this is not the case, it should be possible to use Eq. (2) to determine the effect of discontinuity of flexural reinforcement. Obviously the value of M_u at any section must depend on the amount of flexural reinforcement properly anchored beyond the shear failure surface passing through that section.

Eq. (2), in spite of its simplicity, is basic and leads to a solution of the shear strength of uniformly loaded beams and of beams under combined bending and direct load.

Simple beams under uniformly distributed load

Eq. (2) can be used for cases of multiple or uniformly distributed loading because the shear strength of a section depends only on the moment capacity and the shear span. The shear span at any section is obtained by dividing the actual bending moment by the shear.

In the case of uniform loading, the moment-shear relation is quite different from that of the single concentrated load. The moment varies with the square of the distance from the center of the beam, following a parabolic curve from a maximum at the center to zero at the support.

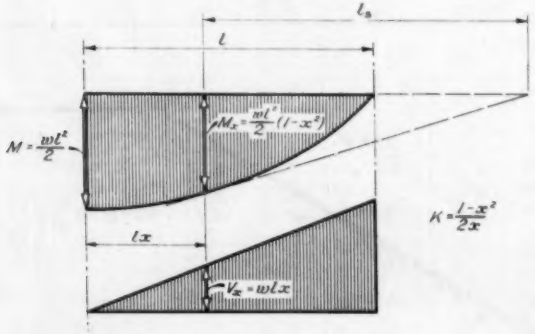


Fig. 11

The shear increases linearly from zero at the center. Referring to Fig. 11, the moment at any section is

$$M_x = \frac{wl^2}{2}(1 - x^2)$$

and the shear is

$$V_x = wxl$$

The shear span l_s is then

$$l_s = \frac{M_x}{V_x} = \frac{wl^2}{2wxl}(1 - x^2) = \frac{(1 - x^2)}{2x}l = Kl$$

in which K equals $(1 - x^2)\ 2\ x$.

A uniformly loaded beam cannot fail in shear at the center of the span where the shear is zero, nor at the support where the shear span is zero. The weakest section must therefore lie at some intermediate point.

Assuming a uniform M_u from end to end of the beam, the ultimate shear strength at any section may be written

$$v_u = 50 + 0.26 \frac{M_u}{d^2} \sqrt{\frac{d}{l_s}} = 50 + 0.26 \frac{M_u}{d^2} \sqrt{\frac{d}{Kl}} \quad \ldots\ldots\ldots\ldots (6)$$

Then the ultimate shear per in. width of beam

$$V_u = v_u jd = 43.7d + 0.2275 \frac{M_u}{d} \sqrt{\frac{d}{Kl}}$$

The actual shear is wxl and if the bending moment at the center is M_u, the uniform load is

$$w = \frac{2 M_u}{l^2}$$

and

$$V = \frac{2 x M_u}{l}$$

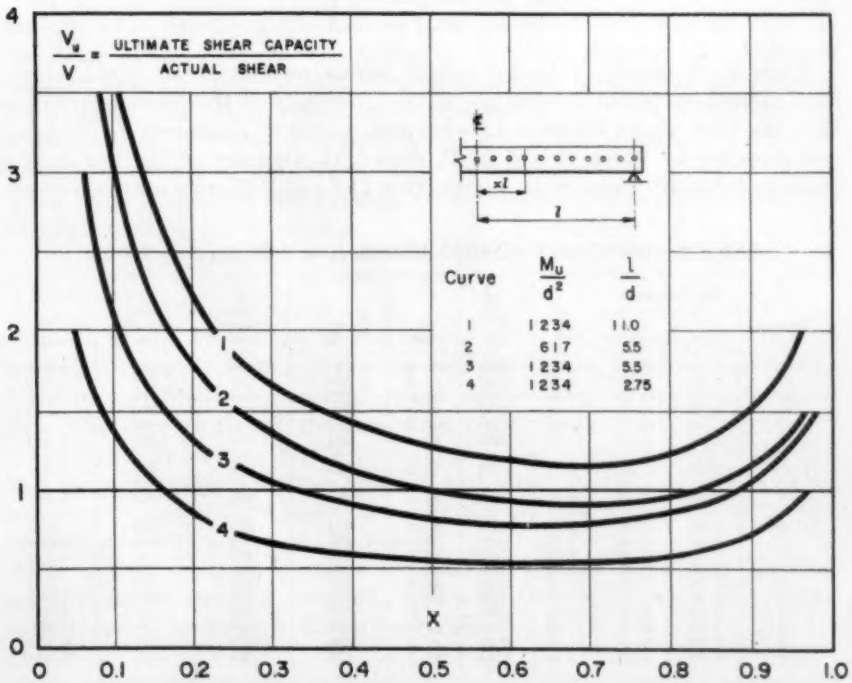


Fig. 12—Uniformly loaded beams. Ratio of ultimate and actual shear

Then the ultimate shear capacity at any section divided by the actual shear is

$$\frac{V_u}{V} = \frac{21.9\ d^2\ l}{x\ M_u\ d} + \frac{0.114}{x}\ \frac{l}{d}\sqrt{\frac{d}{Kl}} \qquad (7)$$

Fig. 12 gives the numerical solution of Eq. (7) for four different combinations of M_u/d^2 and d/l.

The curves show where the beams are deficient in shear strength required to develop the full moment capacity. They indicate that the beams are weakest in shear in the section from six- to seven-tenths of the distance from center to support.

Assuming the critical section to be at $x = 0.7$, the ultimate shear at that point where $l_s = [(1 - x^2)/2x]\ l = 0.365l$ will be from Eq. (6)

$$v_u = 50 + 0.431\ \frac{M_u}{d^2}\sqrt{\frac{d}{l}}$$

This is equal to 0.7 of the ultimate shear at the support which is then

$$v_u = 71.4 + 0.615\ \frac{M_u}{d^2}\sqrt{\frac{d}{l}} \qquad (8)$$

Tests of 18 uniformly loaded simple beams without shear reinforcement were carried out about 2 years ago at the University of Illinois in cooperation with the Ohio River Division Laboratories, Corps of Engineers, U. S. Army and reported by Bernaert and C. P. Siess.[9] A summary of the test data is given in Table 5. Three of the beams (D-9, D-7 and D-6) were over-reinforced.

TABLE 5—UNIFORMLY LOADED BEAMS; b = 6.00 in., d = 9.94 in.

Beam	Reinforcement		$\frac{L}{d}$	f'_c, psi	f_{sy}, ksi	$\frac{M_u}{d^2}\sqrt{\frac{d}{l}}$	v_u (1st. crack), psi	v_{calc} Eq. (9), psi	$\frac{v_u}{v_{calc}}$
	Bars	p, percent							
D–15	3 #4	1.01	8.85	3860	66.8	287	268	225	1.190
14	3 #4	1.01	8.85	4650	67.0	294	289	229	1.265
16	3 #4	1.01	8.85	5760	85.0	372	289	271	1.065
13	3 #4	1.01	11.07	2780	67.3	248	218	204	1.070
12	3 #4	1.01	11.07	4160	67.0	261	—	211	—
17	3 #4	1.01	11.07	5970	85.0	335	236	251	0.940
D– 5*	2 #6	1.55	6.07	3740	52.3	405	306	289	1.060
D– 4	3 #6	2.21	8.85	5020	44.3	412	381	292	1.305
D– 9†	2 #9	3.36	8.85	2790	42.0	442	330	308	1.070
10	2 #9	3.36	8.85	3540	39.0	487	364	333	1.090
11	2 #9	3.36	8.85	5840	48.3	640	462	415	1.110
7†	2 #9	3.36	11.07	2700	42.0	383	337	277	1.215
6†	2 #9	3.36	11.07	3450	45.0	490	337	334	1.010
1	2 #9	3.36	11.07	4450	42.8	498	366	339	1.080
2	2 #9	3.36	11.07	5590	44.5	537	360	360	1.000
8	2 #9	3.36	11.07	6600	57.2	678	378	436	0.865
3	2 #9	3.36	13.28	4820	45.5	484	372	331	1.120
18	2 #9	3.36	15.49	4240	49.5	456	293	316	0.926

*For Beam D–5 only, d = 10.88 in.
†Beams over-reinforced

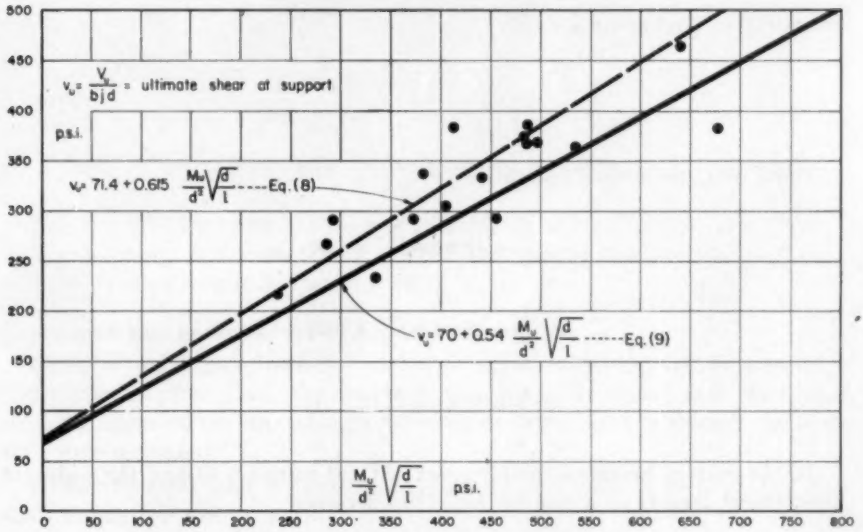

$v_u = \frac{V_u}{bjd}$ = ultimate shear at support
p.s.i.
$v_u = 71.4 + 0.615 \frac{M_u}{d^2}\sqrt{\frac{d}{l}}$ ----Eq. (8)
$v_u = 70 + 0.54 \frac{M_u}{d^2}\sqrt{\frac{d}{l}}$ ------Eq. (9)
$\frac{M_u}{d^2}\sqrt{\frac{d}{l}}$ p.s.i.
0 50 100 150 200 250 300 350 400 450 500 550 600 650 700 750 800
0 50 100 150 200 250 300 350 400 450 500

Fig. 13—Beams under uniform load

The average location of the initial diagonal tension crack at the tensile steel was at 78 percent of the distance from the center of span to the support. The shape of the V_u/V curves in Fig. 12, and their flatness near minimum value, explain why there should be some accidental variation in the location of the crack. The crack crossed the neutral axis of the beam within the zone of minimum V_u/V where x is from 0.6 to 0.7, which appears to verify the theoretical location of the critical shear section.

The value of shear at first diagonal tension cracking shows good agreement with the equation

$$v_u = 70 + 0.54 \frac{M_u}{d^2}\sqrt{\frac{d}{l}} \qquad (9)$$

as shown in Fig. 13. These 18 beams were 6 in. wide and the fact that all but one of them (D-5) had reinforcing steel crowded to a greater extent than might be permitted in a practical design may account for some variation in their behavior even though the theoretical bond stress was not excessive.

The variation between Eq. (8) and (9) may also be due in part to the fact that the relation between the moment and shear curves is different under concentrated and uniform loads. Analysis of more tests is needed to determine the best constants for use in practice.

The critical beam proportions required to balance shear and moment capacity can be obtained from Eq. (9) by equating shear strength with the shear required to develop M_u at the center

$$v_u = 70 + 0.54 \frac{M_u}{d^2}\sqrt{\frac{d}{l}} = \frac{2\,M_u}{jdl}$$

from which, approximately,

$$\sqrt{\frac{l}{d}} = \sqrt{\left(\frac{M_u}{260d^2}\right)^2 + \frac{M_u}{31d^2}} - \frac{M_u}{260d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

From this, the critical length is

$$\frac{L}{d} = 13 \text{ when } \frac{M_u}{d^2} \text{ is } 500$$

$$\frac{L}{d} = 18 \text{ when } \frac{M_u}{d^2} \text{ is } 1000$$

$$\frac{L}{d} = 21 \text{ when } \frac{M_u}{d^2} \text{ is } 1500$$

In the case of balanced reinforcement M_u/d^2 equals $f_c'/3$ and the value of the critical length as given by Eq. (10) becomes:

$$\sqrt{\frac{l}{d}} = \sqrt{\left(\frac{f_c'}{780}\right)^2 + \frac{f_c'}{93}} - \frac{f_c'}{780} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10a)$$

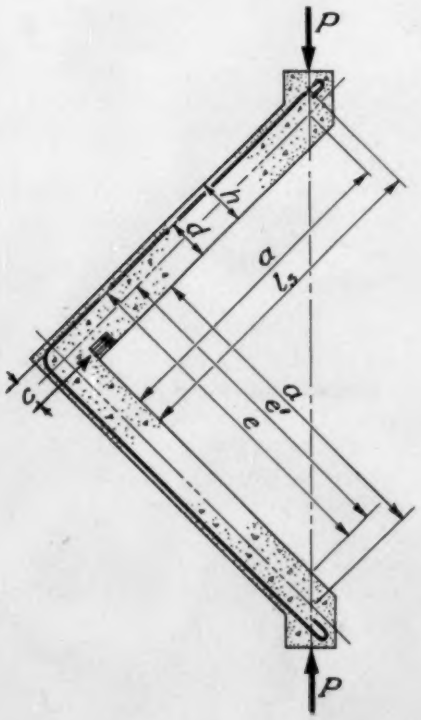


Fig. 14—Knee frames

The critical length and shear strength for various values of f_c' is then:

When $f_c' = 2000$, $L/d = 15.0$, $v_u = 201$, or $0.100\, f_c'$

$f_c' = 3000$, $L/d = 18.0$, $v_u = 250$, or $0.083\, f_c'$

$f_c' = 4000$, $L/d = 20.5$, $v_u = 294$, or $0.073\, f_c'$

It is seen that the value of v_u as given by Eq. (9) is much higher for the beam under uniformly distributed load than in the case of the concentrated load and the critical length is also greater.

Frame members under flexure and axial compression

When an axial compression is added to an under-reinforced simple beam under concentrated load, the moment capacity is increased and the shear strength increased correspondingly. The effect is the same as though the steel area were increased.

When the axial load is tension the moment capacity is reduced and the shear strength is also reduced.

The moment capacity of a beam with no compression steel under bending and direct compression, when the eccentricity is large, may be calculated from the plastic theory equation:[7]

$$P = \left[\frac{A_s f_y}{b}\right] \frac{e + d - \sqrt{(e-d)^2 + \dfrac{2\,e\,A_s\,m}{b}}}{e - d + \sqrt{(e-d)^2 + \dfrac{2\,e\,A_s\,m}{b}}} \qquad \text{.....(11)}$$

The moment capacity, $M = Pe$, and the shear strength will be given by Eq. (2).

Slightly better results are obtained by using the value of c as in Eq. (2a). The value of c (Fig. 14) can be calculated from the equation:

$$c = d - \frac{P + d\,p\,f_y}{1.7\,f_c'}$$

If it were assumed that the resultant of compression force in the concrete is at the face of the beam, the effect of the addition of the axial compression would be to increase the moment capacity of the beam in the ratio $e/(e - d)$. Eq. (2) could then be applied to get approximate results by introducing the term $e/(e - d)$ and using the value of M_u for the beam without axial load as follows:

$$v_u = 50 + 0.26\,\frac{M_u}{d^2}\left(\frac{e}{e-d}\right)\sqrt{\frac{d}{l_a}} \qquad \text{.....(12)}$$

TABLE 6—KNEE FRAMES

Specimen	p, %	f_s, ksi	f_c', psi	b, in.	d, in.	l_s, in.	e, in.	Reinforcing bars	$\frac{M_u}{d^2}\left(\frac{e}{e-d}\right)\sqrt{\frac{d}{l_s}}$, psi	v_u, psi	v_{calc}, psi	$\frac{v_u}{v_{calc}}$
a = 21.60												
F21 B2	1.88	54.6	1470	12.0	14.50	29.85	27.85	2 #6 4 #7	713	195	235	0.830
B2R	1.88	54.6	2040	12.0	14.50	29.85	27.85	2 #6 4 #7	990	255	307	0.831
E2	0.34	65.8	1750	12.0	14.75	29.85	28.10	2 #5	307	228	130	1.754
B4	1.88	54.8	4310	12.0	14.50	29.85	27.85	2 #6 4 #7	1285	367	384	0.956
B4R	1.88	54.8	4230	12.0	14.50	29.85	27.85	2 #6 4 #7	1281	348	383	0.909
C4	1.59	62.9	3860	12.0	14.50	29.73	27.98	2 #5 5 #6	1226	325	369	0.881
C4R	1.56	64.8	4480	12.0	14.75	29.73	27.98	2 #5 5 #6	1305	365	389	0.938
D4	1.22	65.6	4570	12.0	14.50	29.73	27.98	7 #5	1038	325	320	1.016
E4	0.80	62.6	4450	12.25	15.00	29.85	28.35	2 #5 2 #6	707	308	234	1.316
F4	0.75	62.0	4350	12.0	14.50	29.85	27.85	3 #6	635	323	215	1.502
G4	0.35	65.5	4420	12.0	14.50	29.85	27.85	2 #5	322	251	134	1.873
A6	2.36	54.6	7020	12.0	14.50	29.85	27.85	3 #7 3 #8	1670	418	484	0.864
B6	1.88	54.8	6520	12.0	14.50	29.85	27.85	2 #6 4 #7	1355	418	402	1.040
a = 38.60												
F38 B2	1.91	54.2	1800	12.0	14.25	46.85	44.60	2 #6 4 #7	486	167	176	0.949
E2	0.50	56.3	2050	12.0	14.50	46.85	44.85	2 #6	212	107	105	1.019
B4	1.84	55.9	4550	12.0	14.75	46.80	45.16	2 #6 4 #7	750	251	245	1.024
D4	1.32	53.4	3900	12.0	15.00	46.80	45.41	2 #5 3 #7	535	239	189	1.265
E4	0.91	53.4	4660	12.0	14.88	46.85	45.23	1 #6 2 #7	384	213	150	1.420
A6	2.91	52.8	6610	12.0	14.00	46.85	44.35	3 #7 4 #8	1062	337	326	1.034
B6	1.81	54.9	6030	12.0	15.00	46.85	45.35	2 #6 4 #7	758	247	247	1.000
a = 55.60												
F55 B2	1.88	54.3	1720	12.0	14.50	63.85	61.85	2 #6 4 #7	356	139	143	0.972
E2	0.52	64.6	2000	12.0	14.50	63.85	61.85	3 #5	189	93	99	0.932
A4	1.98	58.8	3830	12.13	14.63	66.67	62.17	2 #7 3 #8	598	218	205	1.063
B4	1.81	55.7	4280	12.0	15.00	63.85	62.35	2 #6 4 #7	555	180	194	0.928
D4	1.45	62.7	3710	12.13	15.00	66.67	62.54	3 #5 4 #6	498	178	179	0.994
E4	0.94	61.3	4110	12.13	15.25	64.04	62.41	4 #6	342	153	139	1.101
A6	3.32	55.0	6100	12.0	13.75	63.85	61.10	4 #7 4 #8	902	294	285	1.032
B6	1.88	54.5	6340	12.0	14.50	63.85	61.85	2 #6 4 #7	577	209	200	1.045
a = 70.00												
F70 B2	1.91	55.5	2090	12.0	14.25	78.25	76.00	2 #6 4 #7	366	134	145	0.924
A4	2.16	54.5	4210	12.0	14.25	78.07	76.19	2 #5 4 #8	515	208	184	1.130
A6	3.34	51.4	5610	12.0	14.50	78.25	76.25	6 #9	750	255	245	1.041
a = 84.00												
F84 B4	1.84	55.3	4300	12.0	14.75	92.20	90.55	2 #6 4 #7	418	189	159	1.189
a = 113.00												
F113 B4	1.87	54.5	3770	12.0	14.50	121.38	119.13	2 #6 4 #7	338	137	138	0.992

or better, using the value of c

$$v_u = 50 + 0.30 \frac{M_u}{d^2}\left(\frac{e}{e-c}\right)\sqrt{\frac{c}{l_s}} \quad \text{.....(12a)}$$

Fig. 15 shows the application of Eq. (12) and (12a) to the Illinois frame tests.[8] They show good agreement except for the 21-in. frames indicating that a more accurate method must be applied to cases with small eccentricity (when e is less than about 2 d). The test data are given in Table 6. The equations have also been applied with good results to tests of unsymmetrical frames in which the angle of the thrust varied.

The case with small e/d ratio needs further study.

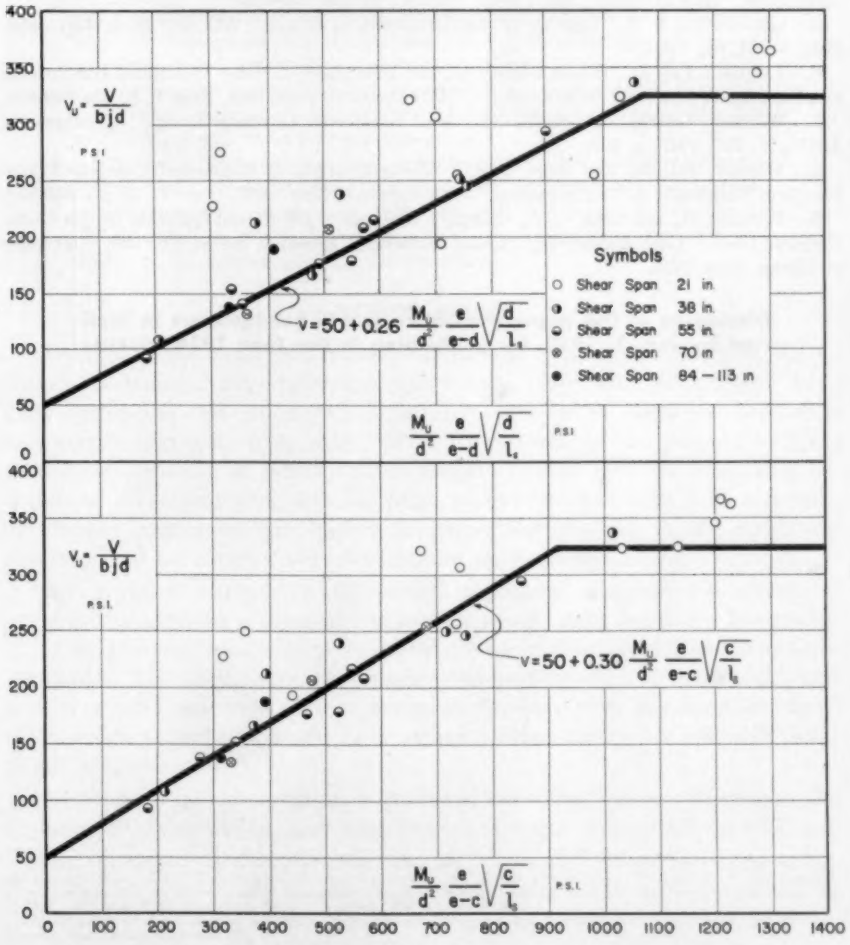

$V_u = \frac{V}{bjd}$
P.S.I.
$v = 50 + 0.26 \frac{M_u}{d^2} \frac{e}{e-d}\sqrt{\frac{d}{l_s}}$
$\frac{M_u}{d^2} \frac{e}{e-d}\sqrt{\frac{d}{l_s}}$
P.S.I.
Symbols
Shear Span 21 in.
Shear Span 38 in.
Shear Span 55 in.
Shear Span 70 in.
Shear Span 84 — 113 in.
$V_u = \frac{V}{bjd}$
P.S.I.
$v = 50 + 0.30 \frac{M_u}{d^2} \frac{e}{e-c}\sqrt{\frac{c}{l_s}}$
$\frac{M_u}{d^2} \frac{e}{e-c}\sqrt{\frac{c}{l_s}}$
P.S.I.
0 50 100 150 200 250 300 350 400
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400

Fig. 15—Knee frames, unit shear at diagonal tension cracking

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Title No. 54-16

Lightweight-Aggregate Concrete for Structural Use*

By J. J. SHIDELER†

SYNOPSIS

Describes tests employed and results obtained in an investigation of properties of lightweight-aggregate concrete of structural quality. Eight lightweight aggregates and one normal weight sand-and-gravel aggregate are included. The total program includes tests of plain, conventionally reinforced, and prestressed concrete specimens, but except for pull-out tests of reinforcing bars, only the data on plain concrete are given. Data are reported on concrete mix proportions, compressive and flexural strength, modulus of elasticity, bond, creep, and drying shrinkage. Concrete mixes were designed to produce compressive strengths of 3000 psi and 4500 psi with each aggregate, and 7000 to 10,000 psi with three selected aggregates.

Comparing the various concretes on the basis of equal compressive strengths, normal weight sand-and-gravel concrete showed somewhat superior performance in most tests. In many structural applications, however, this superiority will doubtless be overshadowed by advantages resulting from reduced unit weight of the lightweight-aggregate concretes.

INTRODUCTION

The lightweight aggregate industry is rapidly expanding. Most of the 41 plants producing lightweight aggregates from clays and shales have been built within the last 10 years. Eighteen plants are in operation producing lightweight aggregate from slag. With the growth in the number and the productive capacity of lightweight aggregate plants, and the increasing demands of structural designers for light weight coupled with high strength, lightweight aggregates are finding an important position in the structural concrete field for precast and cast-in-place applications.

This growing interest in lightweight-aggregate concrete for structural purposes has created a demand for reliable design data beyond a knowledge of its weight and compressive strength. With this in mind, the present investigation was undertaken to compare the performance of lightweight and normal weight concretes so that engineers familiar with standardized structural design procedures would have representative values for use with lightweight concrete.

The program included two main groups of tests, the first on concrete having a slump of about 2½ in. and compressive strength from 3000 to 5000 psi.

*Presented at the ACI 53rd annual convention, Dallas, Tex., Feb. 28, 1957. Title No. 54-16 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 4, Oct. 1957, *Proceedings* V. 54. Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Jan. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Member American Concrete Institute, Manager, Products and Applications Development Section, Portland Cement Association, Chicago, Ill.

This type of concrete is required for cast-in-place structural use. In this group of tests mixes were proportioned for 28-day strengths of about 3000 psi and 4500 psi with each of seven lightweight aggregates and one sand-and-gravel aggregate. The lightweight-aggregate concretes and sand-and-gravel concrete are compared on the basis of equal strengths.

The second group of tests covered concrete having zero slump and compressive strength from 7000 to 10,000 psi. Recent advances in concrete technology have led to casting structural members using very stiff concretes of a relatively high cement content, with compressive strengths in excess of 7000 psi. Such concretes are used only in precasting where concreting operations are under careful control. The low-slump concrete is consolidated in the forms by prolonged vibration or shock methods. Particular attention is given to the forms, form oils, and placing procedures to insure smooth and void-free concrete surfaces. These improved techniques will lead to the increased production of standardized structural members which are fabricated, stocked, and catalogued for use by the construction industry.

At present very little concrete of this quality has been used in the United States; however, there appears to be a large field for its use. To obtain structural design data for this class of concrete a limited investigation of its properties has been undertaken with particular emphasis on lightweight-aggregate concrete. Two lightweight aggregates and one sand-and-gravel aggregate were included in this high-strength concrete series.

The testing program was planned to include the following: physical properties of the aggregates; properties of plain concrete; bond, shear resistance, and ultimate load capacity of reinforced beams; and bond and stress transfer characteristics of prestressed concrete. Only a portion of the total program has been completed. Data on concrete mixes, compressive strength, elastic properties, creep, drying shrinkage, bond, and flexural strength of concrete are included in this report. Various lightweight concretes, even those containing aggregates produced by similar processes, show rather large variations in structural properties. It is therefore important that individual producers conduct investigations to provide reliable design data for specific materials.

TEST PROCEDURE

Aggregate

The eight lightweight aggregates were selected to encompass representative types and methods of manufacture used throughout the United States. These aggregates included four shales, a slate, and a clay expanded in rotary kilns; a shale expanded by the sintering process; and an expanded blast-furnace slag. They were obtained from a wide range of geographical locations.

All aggregates were screen separated into a number of sizes and recombined to a specified grading; they do not necessarily represent the commercial product of the producer. The aggregates (Fig. 1), referred to by number, are briefly described as follows:

Aggregate 1—an experimental product made from shale expanded in a rotary kiln. The raw material is crushed and screened to four sizes and these are bloated in the kiln separately. Particles are rounded, have a relatively smooth and impervious shell. Part of the material passing the No. 50 sieve is obtained by crushing.

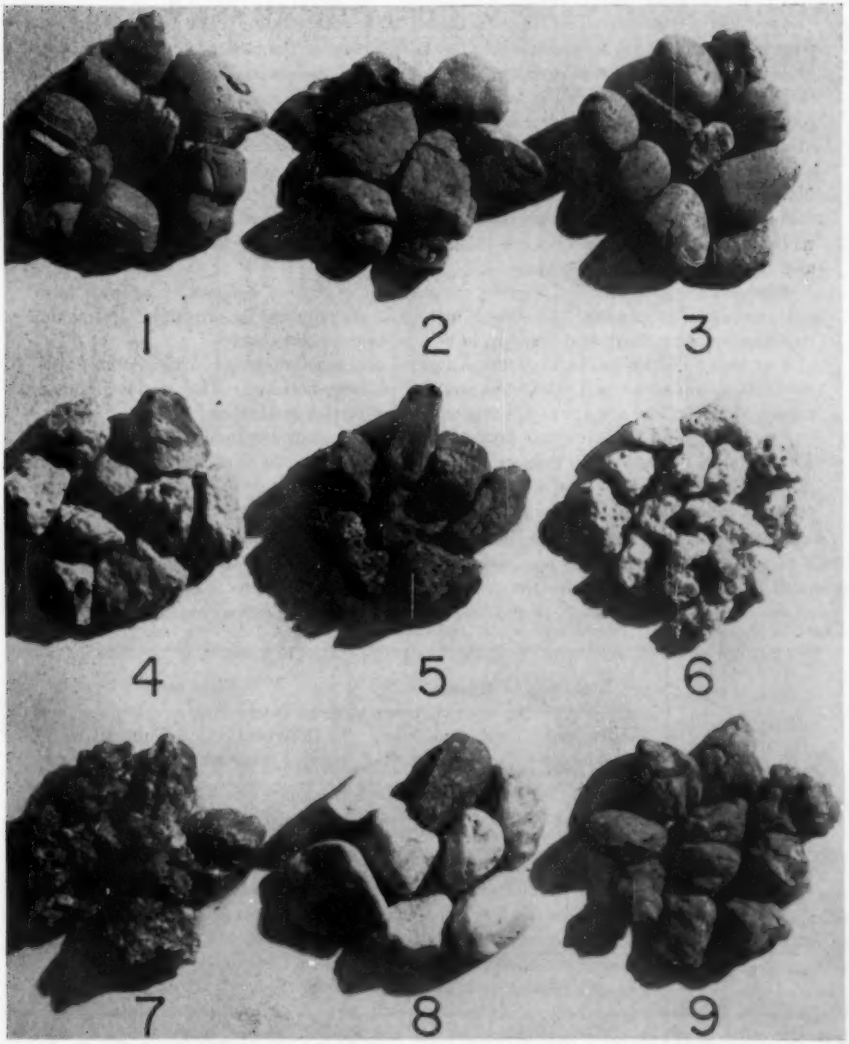

Fig. 1—Aggregates used in lightweight-aggregate concrete investigation

Aggregate 2—an expanded shale produced in a rotary kiln. The raw material is screened only for maximum size prior to bloating. The coarser sizes are rounded and have a smooth shell similar to Aggregate 1. The fine material, passing the No. 4 sieve, is obtained by crushing the coarser material.

Aggregate 3—an expanded shale similar to Aggregate 1 in appearance and in method of manufacture. The raw material is crushed and screened to four sizes. These individual fractions are passed through separate kilns. All particles are rounded and have a smooth shell down to and including material retained on the No. 100 sieve.

Aggregate 4—an expanded clay produced in a rotary kiln. The raw material is passed through a 1-in. screen and into the kiln. Most of the bloated material from the kiln does not require crushing except to produce additional material passing the No. 50 sieve. The bloated material is very fine grained, but the individual particles are not rounded and they do not have a shell.

Aggregate 5—an expanded slate produced in a rotary kiln. The coarser particles are angular and porous, but are not as sharp as Aggregates 6 and 7. Material passing the No. 4 sieve is obtained by crushing.

Aggregate 6—an expanded slag produced by spraying a controlled amount of water on molten slag. Most of the sizes are obtained by crushing; all particles are very angular and have a porous surface texture.

Aggregate 7—produced by burning on a sintering grate a carbonaceous shale from anthracite coal processing. All sizes of aggregate are obtained by crushing. Individual particles are very sharp and angular, have a porous surface texture.

Aggregate 8—Elgin sand and gravel, a natural uncrushed material. The gravel is well-rounded, and is about half calcareous and half siliceous material. The sand is predominately quartz. The aggregate has a good service record in northern Illinois.

Aggregate 9—an experimental product made from shale expanded in a rotary kiln. The material was separated into several size fractions prior to bloating and the finished product is generally rounded and sealed. This aggregate was used only in the high-strength concrete program.

Representative pieces of the various aggregates are shown in Fig. 1. These were selected from the coarse aggregate and are pictured at about ¾ natural size. All aggregates were screened to six sand sizes and to two sizes of coarse aggregate. The fine material was recombined, for each individual batch, to the following grading on a clean separation basis:

	Lightweight aggregates		Elgin sand	
Sieve size	Individual percent retained	Cumulative percent retained	Individual percent retained	Cumulative percent retained
8	15	15	18	18
16	25	40	15	33
30	20	60	24	57
50	18	78	30	87
100	12	90	8	95
pan	10	100	5	100
F.M.		2.83		2.90

This grading is about midway between the limits established in ASTM C 330-53T for lightweight aggregates. Fine natural sand was not added to any of the lightweight concrete mixes, although sometimes this has been recommended to improve workability and strength. The coarse aggregate was recombined using equal weights of No. 4 to ⅜ in. and ⅜ in. to ¾ in. Unit weights of the graded materials were determined according to ASTM C 29-42 by dry-loose and dry-rodded methods.

The absorption, specific gravity, and unit weights of the graded materials are recorded in Table 1. The specific gravity and absorption of the coarse aggregate were determined according to ASTM C 127-42. Due to the difficulty of determining the saturated surface-dry condition of the fine aggregate, the "kerosene method"[1] was used in preference to ASTM C 128-42. A 300-g sample of dry fine material was put in a flask and mixed with enough kerosene (about 20 ml) to wet all the surfaces of the aggregate. Water was added and the amount required

TABLE 1—ABSORPTION, SPECIFIC GRAVITY, AND UNIT WEIGHT OF AGGREGATES

Aggregate No.	Sieve size	Absorption, percent			Specific gravity			Unit weight,* lb per cu ft	
					Bulk				
		1 hr	4 hr	24 hr	Oven dry	Saturated surface-dry	Apparent	Dry, loose	Dry, rodded
1	0 – 4	2.6	3.2	4.0	1.81	1.88	1.96	66.1	71.5
	4 – ¾ in.	4.2	5.3	6.7	1.12	1.20	1.21	37.5	41.3
2	0 – 4	2.2	4.1	6.8	1.75	1.86	1.98	52.0	56.2
	4 – ¾ in.	11.3	12.0	14.9	1.17	1.34	1.41	42.1	45.0
3	0 – 4	3.1	3.6	4.8	1.91	2.00	2.10	72.0	77.2
	4 – ¾ in.	6.6	8.0	9.7	1.21	1.33	1.37	43.7	45.4
4	0 – 4	9.6	11.1	13.5	1.76	2.00	2.34	57.8	63.7
	4 – ¾ in.	11.3	12.7	15.6	1.50	1.73	1.96	49.6	54.2
5	0 – 4	3.4	4.4	6.6	1.97	2.10	2.26	62.7	68.6
	4 – ¾ in.	8.1	9.2	11.0	1.30	1.44	1.54	43.9	47.3
6	0 – 4	2.3	2.9	4.0	2.10	2.19	2.30	64.2	70.2
	4 – ¾ in.	6.2	6.6	7.1	1.46	1.56	1.62	44.2	47.3
7	0 – 4	5.6	6.3	7.0	1.91	2.05	2.21	64.0	68.9
	4 – ¾ in.	6.5	7.0	7.3	1.59	1.72	1.82	45.9	48.0
8	0 – 4	—	—	2.2	—	2.65	—	108.2	112.7
	4 – ¾ in.	—	—	2.1	—	2.65	—	100.0	108.2
9	0 – 4	2.6	3.1	4.0	1.81	1.88	1.95	73.0	77.5
	4 – ¾ in.	3.4	3.8	5.2	1.26	1.33	1.35	45.0	49.5

*ASTM C 330-53T.

to bring the water level to the 500-ml calibration mark was noted. The film of kerosene prevented absorption of water into the aggregate during the brief time required for this test.

$$\text{Bulk specific gravity (oven-dry)} = \frac{A}{V - W}$$

A = weight in grams of oven-dry sample in air
V = volume of flask (500 ml)
W = volume of water added to flask, ml

Absorption measurements were made on a companion 300-g oven-dry sample. The sample was put in a flask and water added to cover the aggregate. The sample was stirred and agitated for about 5 min to remove entrapped air. The flask was then filled with water to the 500-ml mark. At periods of 1, 4, and 24 hr the flask was again filled to the reference mark. The volume of the sample V_s can be computed from the weight and specific gravity. The volume of water to fill the flask to the 500-ml mark is merely $500 - V_s$. Any water in addition to the amount required to keep the water level at the 500-ml mark is due to an equal amount being absorbed into the aggregate. By making proper weights of the flask, sample, and water, absorption at any time can be calculated.

The bulk specific gravity on a saturated surface-dry basis was computed from data obtained during the specific gravity and absorption tests just described.

$$\text{Bulk specific gravity (saturated surface-dry)} = \frac{B}{V - W}$$

B = dry weight of sample plus weight of water absorbed in 24 hr

$$\text{Apparent specific gravity} = \frac{A}{V - W - C}, \quad C = \text{volume of water absorbed in 24 hr}$$

Concrete mixes

All lightweight aggregates were air-dry when placed in the concrete mixer. Some of the lightweight aggregates were received dry, and some were dried prior to screening. In field use aggregates vary widely in moisture content. They are all manufactured materials and are initially dry, but generally they are sprinkled to keep down dust and to reduce segregation. Other investigators[1,2] have shown that whether the aggregates are mixed wet or dry has little effect on the compressive strength of the concrete for the range of consistencies and cement contents used. Discussions with representatives of the lightweight aggregate industry led to the conclusion that using dry aggregates was the most reasonable method of operation for the laboratory tests.

A 1¾-cu ft tub-type mixer was used. The aggregates and about 2/3 of the mixing water and air-entraining agent were mixed 2 min; the cement and remaining water and agent were then added and mixing continued for 3 min more. The amount of air-entraining agent required to produce 5 percent air showed a wide variation, due in large part to the quantity of agent absorbed by the aggregate. More uniform amounts of agent would probably have been required if all of the agent had been added with the final 1/3 of the mixing water. A blend of four commercial Type I cements was used in all tests.

3000- and 4500-psi series—Concretes having 28-day compressive strengths of 3000 psi and 4500 psi were desired. Because the aggregates were mixed from a dry condition, ordinary mix proportioning procedures would have been difficult to apply. The principal difficulty was the determination of the amount of absorbed water. The absorption of some of the lightweight aggregates was high compared to that of sand and gravel, and absorption continued for several days. Values of 24-hr absorption were between 4 and 15 percent for the different aggregates, and the 1-hr absorption varied from 30 to 70 percent of the 24-hr absorption. Due primarily to this high and variable absorption, reliable values of specific gravity were difficult to obtain.

Concrete mixes were proportioned on the basis of cement content, with the water content adjusted to produce a 2½-in. slump, and with a minimum amount of fine aggregate (passing No. 4 sieve) to produce adequate workability. Preliminary mixes were made with two cement contents and three different percentages of fine material. Proportions were then selected to produce concrete of the required strength and workability.

In actual mix calculation, weights of materials per cubic yard were obtained from the batch weights and unit weight of the plastic concrete. The percent of purposely entrained air was calculated as described below:

> The weight of each of the materials, the unit weight of the fresh concrete, and the yield were known or calculated for each batch of concrete without an air-entraining agent. The volumes occupied by the cement and total water were determined. The remaining volume of the batch was occupied by a known weight of aggregate. By assuming that the absolute volumes of the coarse and fine aggregates were proportional to the dry, loose unit weights, specific gravities of these two sizes of materials were computed. This assumption is not precisely true, but this does not affect the usefulness of the method. These "specific gravity factors" were then used to compute the absolute volume of aggregate in the air-entrained concrete mixes and to arrive at the volume of purposely entrained air. The air content computed in this manner was about 1 percent greater than that indicated by the difference in unit weights of air-entrained and non-air-entrained concrete. In later mixes the volumetric-type air meter (ASTM C 173) was used with good results.

7000- to 10,000-psi series—Trial mixes were made with each of the aggregates to obtain an estimate of mix proportions required for the desired strength and workability. No air-entraining agent was used in the high-strength mixes. The consistency of this zero-slump concrete was measured by use of the "Vee-Bee" consistometer[3] as well as by slump tests. The Vee-Bee apparatus was also used as a vibrating table to consolidate cylinders and other test specimens. This type of concrete was too stiff to be easily consolidated by means of an internal vibrator but was readily compacted on the vibrating table.

The trial mixes indicated that concrete with a compressive strength of 7000 psi could be obtained with Aggregate 9 and 9000-psi concrete could be obtained with Aggregate 4 at a cement content of 10 bags per cu yd. The cement content of 10 bags per cu yd was selected arbitrarily as the upper limit that would generally obtain for practical and economic usage.

Trial mixes were also made with sand-and-gravel aggregate with cement contents between 7 and 10 bags. In these initial tests the sand-and-gravel aggregate was dry at the time it was introduced into the mixer. The 28-day compressive strength increased from 6300 to 7000 psi as the cement content was increased from 7 to 10 bags. However, later tests showed that the compressive strength of the sand-and-gravel concrete was greatly affected by the initial moisture content of the aggregate and by the curing conditions. A brief series of tests with sand-and-gravel concrete indicated that the 28-day strength of richer mixes was increased about 15 percent by ponding* the cylinders immediately after casting and that the strength was further increased by using soaked aggregates. As a result of these improved techniques the compressive strength of the sand-and-gravel concrete exceeded 9000 psi at a cement content of 10 bags per cu yd.

Soaking the lightweight aggregates used in these tests and ponding the lightweight concrete cylinders generally had little effect on the 28-day compressive strength. Consequently, the lightweight aggregates were introduced dry and the cylinders were covered with steel plates immediately after casting. The sand and gravel was soaked 24 hr prior to mixing and the concrete cylinders were covered with a film of water within a few minutes after they had been cast. Only about 50 ml of water was absorbed per cylinder in the 24-hr ponding period.

Only five mixes were involved in the program. One mix with each aggregate was designed for 7000 psi, and mixes with Aggregate 4 (lightweight) and sand-and-gravel Aggregate 8 were designed for 9000 psi. The fine aggregate was 25 to 35 percent of the total aggregate for the various materials. No air-entraining agent was used. All mix data are recorded in Table 2.

Compressive strength

Compressive strength determinations were made on 6 x 12-in. cylinders. Some specimens were continuously fog cured and some were dried at 50 percent relative humidity and 73 F after 7 days of fog curing. Compressive strength values were obtained at several ages from 1 to 365 days. Prior to testing, the cylinders in the 3000- and 4500-psi series were capped with a proprietary compound containing sulfur and a filler material. Initially the cylinders in the high strength series were capped with this compound, but most of the specimens were prepared for testing by lapping the ends. There was some indication that recorded strengths were higher and strength variations between companion cylinders were smaller for specimens with ground ends. Specimens that were stored at 50 percent relative humidity were tested in the dry condition.

Generally two cylinders were provided for each test condition. However, control cylinders were cast from every batch so that 10 to 15 cylinders representing about ten batches for each aggregate and strength of concrete were tested in dry condition at 28 days. These cylinders were cast over a period of several months. The coefficient of variation for each group of 10 to 15 companion cylinders in the high strength series ranged from 2.18 to 4.74 percent, averaging 3.23 percent. The coefficient of 4.74 percent was obtained on the 7000-psi sand-and-gravel concrete and included both capped and ground cylinders.

Modulus of elasticity

Static modulus of elasticity was determined for all cylinders used for compressive strength tests. Axial strain measurements were made by a dial mounted in a frame secured to the concrete specimen. An additional dial measured lateral deformation for determining Poisson's ratio. The secant modulus of elasticity at $0.3\,f_c'$ is reported.

Dynamic (resonant) modulus of elasticity and the modulus of rigidity were determined as described in ASTM C 215-52T. Poisson's ratio was also computed from these dynamic measurements.

*"Ponding" here refers to covering the concrete with a film of water.

TABLE 2—CONCRETE MIX DATA

Aggregate No.	Fine aggregate, percent*	Quantities per cu yd of concrete					Percent air, computed	Unit weight of plastic concrete, lb per cu ft	V-B deg	Slump, in.	Cylinders†	
		Water, lb	Cement		Aggregate						Unit weight, lb per cu ft	Absorption, percent by volume
			Bags	lb	Fine, lb	Coarse, lb						
3000-psi series												
1	40	359	5.45	512	858	704	6.9	90.1	—	2¾	88.18	13.82
2	50	441	6.28	590	780	643	6.4	90.9	—	2½	88.50	17.24
3	45	327	5.07	477	945	742	6.0	92.3	—	2¾	90.03	12.94
4	55	497	4.43	416	1026	722	6.3	98.5	—	2¼	90.68	17.80
5	55	417	5.81	546	1071	591	7.7	97.2	—	2½	92.90	16.32
6	55	444	6.66	626	1195	655	5.5	108.2	—	2¼	107.01	19.18
7	55	459	6.39	601	1184	687	5.7	108.6	—	2½	106.90	18.76
8	48	226	3.88	366	1592	1726	5.4	144.8	—	1½	—	—
4500-psi series												
1	35	371	8.18	769	704	714	3.2	94.7	—	2¾	94.27	12.82
2	45	445	8.18	769	686	691	5.9	95.9	—	2¾	92.82	16.78
3	40	363	7.34	690	823	791	6.0	98.8	—	2¾	96.94	13.20
4	50	487	6.00	564	910	770	5.6	101.1	—	2¾	96.60	16.38
5	50	429	7.79	732	963	647	6.8	102.6	—	2½	100.95	14.27
6	50	420	8.35	785	1036	693	4.0	109.1	—	2¾	109.05	16.30
7	50	441	8.15	766	1036	735	5.9	110.3	—	2¾	109.89	16.20
8	39	220	4.82	453	1284	2008	4.6	147.0	—	2½	—	—
7000-psi series												
9	30	330	10.32	970	651	930	0	106.7	14	½	—	—
4	35	462	6.88	647	715	1117	0	109.1	10	¼	105.0	15.3
8	25	295	5.93	557	824	2527	0	155.4	8	¼	153.5	7.5
9000-psi series												
4	35	479	10.33	971	638	999	0	114.3	13	¼	110.2	14.2
8	25	330	10.11	950	727	2177	0	154.8	16	¼	152.5	4.0

*Percent of material passing No. 4 sieve by dry loose volume
†Properties determined by ASTM C 330-53T

Flexural strength

Modulus of rupture for 6 x 6 x 30-in. beams was determined in accordance with ASTM C 78-49, using an 18-in. span and third-point loading. In the 3000- and 4500-psi series beams were continuously fog cured and were tested at 7, 28, and 90 days, and companion beams were fog cured 7 days and then stored at 50 percent relative humidity until tested at 28 days. In the high-strength series, specimens were tested in flexure at ages to 6 months under both moist curing and drying conditions. The compressive strength of modified cubes from all beams was also determined (ASTM C 116-49). Two beams for each test condition provided four flexure tests and four modified cubes.

Pull-out tests

Tests to determine the bond developed between the concrete and reinforcing steel were conducted according to ASTM C 234-49T with the exception that larger bars were used in the high strength concrete. Reinforcing bars conforming to ASTM A 305 were embedded in three different positions; #6 bars were used in the 3000- and 4500-psi concretes and #9 bars were used in the high strength concrete. One bar was embedded horizontally 4½ in. from the bottom (to the center of the bar) and one was embedded 13½ in. from the bottom of a 9 x 9 x 18-in. specimen. One bar was embedded vertically in a 9-in. cube. The larger specimen was broken into two 9-in. cubes at 7 days. All specimens were fog cured 7 days and then dried 21 days at 50 percent relative humidity. Slip was measured at both the loaded and free ends of the bar.

Creep tests

Creep tests under sustained loading were conducted on 6 x 12-in. concrete cylinders containing each of the aggregates. Specimens from two classes of concrete (3000 and 4500 psi) were loaded to 600 psi after 7 days of fog curing. Companion specimens were loaded after 7 days fog curing and 21 days drying at 50 percent relative humidity. Additional specimens were also subjected to loads of 600 and 2000 psi after storage at 50 percent relative humidity for approximately 18 months. The high-strength concrete was loaded to 2000 psi under similar conditions. All tests were conducted at 50 percent relative humidity and 73 F, and additional representative cylinders, not loaded, were provided for drying shrinkage measurements. To assure good alignment in the loading frames and uniform distribution of load, all specimens were capped on both ends in a capping jig. Three cylinders, each containing a different aggregate, were loaded in one testing frame; two cylinders were provided for each test condition. The load was applied by means of a hydraulic jack (Fig. 2), equipped with a calibrated pressure gage, working against the top fixed plate. Nuts were then tightened by hand against the second plate and the jack was removed. A compression spring in the bottom of the frame maintained the applied load. The load was checked with the hydraulic jack and calibrated pressure gage equipment at frequent intervals during the first few weeks and about once a month thereafter.

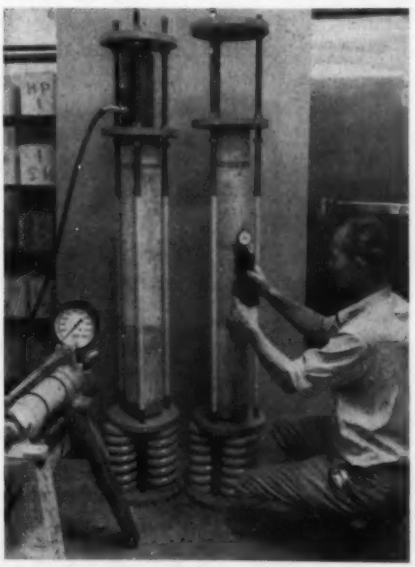

Fig. 2—Creep specimens were loaded by means of hydraulic jack; length measurements were taken with a Whittemore strain gage

Length changes of the cylinders were measured by a 10-in. Whittemore strain gage. Brass plugs cast into the cylinders were fitted with stainless steel inserts containing a small hole for the gage points. Three gage lines spaced at 120 deg around the circumference were provided on each cylinder.

Alkali-aggregate reaction

Concrete prisms (2 x 2 x 11¼ in.) were made with each of the aggregates to determine the potential expansive alkali reactivity of the cement-aggregate combinations. The prisms were stored over water at 100F and measurements of weight and length change were made at frequent intervals. The alkali content of the cement was 0.63 percent expressed as Na_2O.

TEST RESULTS AND DISCUSSION

Concrete suitable for structural use was obtained with each of the aggregates. However, a wide variation in cement content was required to produce the desired strength, and concretes having the same slump showed a great variation in workability in the lower-strength series.

The preliminary mix design relationships between cement content, compressive strength, gross water-cement ratio, and unit weight of both air-entrained and non-air-entrained concretes in the two lower-strength series are shown in Fig. 3. The curves summarize the data obtained from about 20 mixes with each aggregate covering compressive strengths varying from 1500 to 6000 psi. Although the amounts of fine aggregate were varied over a range of 10 percent for each aggregate, these relationships are not included in the figures. The non-air-entrained mixes required at least 5 percent more fine material than the air-entrained concretes. The amounts of fine aggregate required to produce adequate workability in the final air-entrained mixes are listed in Table 2. Materials per cubic yard of concrete, calculated from the unit weight of the wet concrete used in the final mixes, are also given in Table 2. The amount of fine material (passing No. 4 sieve) had little effect on the compressive strength in the range of practical mixes. The three expanded shales produced in rotary kilns (Aggregates 1, 2, and 3) required less fine material than the others. The mixes in the 4500-psi series required about 5 percent less fine aggregate than the 3000-psi series mixes for the same workability.

Aggregate 4, an expanded clay, produced considerably higher strengths at a given cement content than any of the other lightweight aggregates. This aggregate also had the highest absorption and highest gross water requirement. Aggregate 6, an expanded slag, required the highest cement content to produce a given strength.

The unit weights of the various lightweight-aggregate concretes in the lower-strength series ranged from 90 to 110 lb per cu ft compared to 146 lb for sand-and-gravel concrete. In the 7000- to 10,000-psi series the unit weights of the lightweight concretes ranged from 106.7 to 114.3 lb per cu ft compared to 155 lb for sand-and-gravel concrete. The unit weight of the fresh concrete and also the unit weights of 6 x 12-in. cylinders fog cured 7 days and dried at 50 percent relative humidity for 21 days are given in Table 2. Expanded

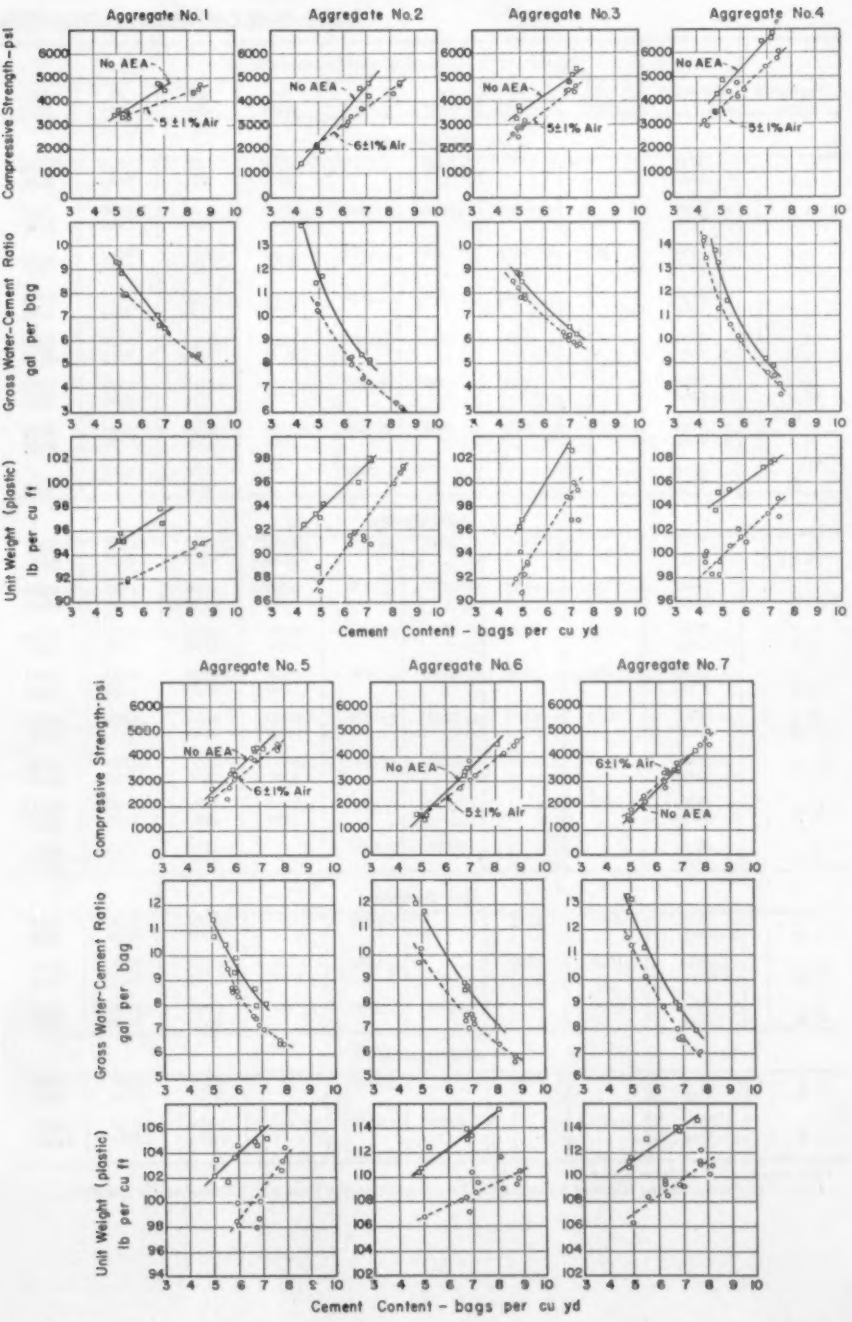

Fig. 3—Concrete mix proportioning data, lightweight-aggregate concrete

TABLE 3—COMPRESSIVE STRENGTH

Aggregate No.	Storage*	Compressive strength, psi							
		1 day	2 days	3 days	7 days	14 days	28 days	90 days	180 days
3000-psi series									
1	Wet			1550	2270	2880	3350	3780	3940
	Dry					2920	3330	3380	3280
2	Wet		1040	1250	1890	2560	3150	3700	3910
	Dry					2340	2950	3560	3400
3	Wet		950	1290	1870	2420	2910	3410	3690
	Dry					2380	2830	3220	3200
4	Wet		850	1090	1620	2220	3020	3820	3980
	Dry					2260	3150	3490	3290
5	Wet		920	1280	1650	2460	2820	3420	3700
	Dry					2140	2550	2640	2770
6	Wet		810	1080	1650	2190	2760	3460	3760
	Dry					2160	2740	2880	2910
7	Wet		970	1170	1850	2450	3050	3940	4190
	Dry					2380	3100	3720	3540
8	Wet				2050		3060	3540	
	Dry						3220		3660
4500-psi series									
1	Wet		2410	2900	3730	4150	4440	4860	5060
	Dry					4150	4460	4620	4340
2	Wet	1150	1850	2270	3170	3760	4300	4850	4600
	Dry					3990	4220	4700	4440
3	Wet	1310	2260	2750	3600	4100	4460	5050	5040
	Dry					4360	4730	4980	5020
4	Wet	780	1400	1900	2880	3720	4540	4990	5380
	Dry					3680	4650	5290	5240
5	Wet	1000	1810	2260	3270	3700	4350	4680	5250
	Dry					3820	4440	—	4700
6	Wet	870	—	1920	2800	3650	4150	4750	4780
	Dry					3540	3980	4380	4100
7	Wet	1050	—	2240	3220	3710	4480	5150	5400
	Dry					3890	4550	5180	5150
8	Wet			2030	3080		4210	4930	5220
	Dry						4470	5000	4760
7000-psi series									
9	Wet	4040	5210		6220	—	6580	7100	7290
	Dry						7060	6920	7200
4	Wet	1520	2600		4980	—	6680	7700	8010
	Dry						7110	8150	8340
8	Wet	1400		3480	5160	—	6990	7700	8470
	Dry						7560	8420	8300
9000-psi series									
4	Wet	2870	4530		6960	—	8360	8820	8800
	Dry						8370	8780	8620
8	Wet	4070		6980	8430	—	9470	10530	11350
	Dry						10570	10280	12000

*Wet—continuously moist cured.
Dry—7-day moist curing followed by storage at 50 percent relative humidity. Tested in dry condition.

AND MODULUS OF ELASTICITY

	Static modulus of elasticity, $E_s \times 10^{-6}$ psi								
365 days	1 day	2 days	3 days	7 days	14 days	28 days	90 days	180 days	365 days
3000-psi series									
4000			1.42	1.45	1.74	1.69	1.92	2.12	2.16
3280					1.70	1.60	1.50	1.56	1.54
4300		1.04	1.05	1.32	1.54	1.70	1.88	2.22	2.07
3320					1.42	1.48	1.50	1.55	1.42
3720		1.26	1.51	1.66	1.83	1.89	2.03	2.18	2.27
3200					1.63	1.83	1.84	1.82	1.88
4060		0.83	1.07	1.21	1.46	1.52	1.81	1.93	1.94
3490					1.46	1.55	1.54	1.58	1.56
3950		1.07	1.41	1.68	1.69	1.86	2.21	2.22	2.26
2570					1.49	1.64	1.71	1.65	1.46
4050		1.12	1.49	1.96	2.14	2.37	2.68	2.85	2.93
2860					2.09	2.18	2.12	1.97	1.88
4570		1.24	1.40	1.81	1.97	2.18	2.56	2.74	2.82
3470					1.82	2.17	1.96	1.87	1.79
—				2.64		3.20	4.00		—
						3.53		3.81	
4500-psi series									
5030		1.56	1.60	1.88	2.10	2.07	2.18	2.26	2.42
4640					1.98	1.96	1.84	1.76	1.90
5050	1.06	1.34	1.33	1.65	1.74	1.97	2.03	2.29	2.31
4650					1.72	1.72	1.74	1.74	1.64
5140	1.38	1.75	1.77	1.98	2.16	2.22	2.41	2.54	2.60
4920					2.13	2.10	2.25	2.16	2.18
5390	0.86	1.17	1.26	1.45	1.63	1.84	2.00	2.21	2.17
5440					1.56	1.79	1.89	1.91	1.87
5280	1.19	1.52	1.57	1.95	2.06	2.19	2.46	2.62	2.56
4500					2.01	2.04	2.20	2.20	2.06
4980	1.19	—	1.95	2.35	2.64	2.83	3.10	3.11	3.09
3920		—			2.48	2.53	2.50	2.40	2.34
6100	1.24		1.96	2.18	2.45	2.62	2.94	3.18	3.12
4950					2.42	2.45	2.40	2.44	2.18
5340			2.43	3.06		3.43	4.58	5.01	5.33
4700						3.84	4.39	4.25	4.18
7000-psi series									
7470	2.46	2.74	—	3.01	—	3.02	3.10	3.24	3.41
7160					—	2.91	2.88	2.83	2.95
8310	1.16	1.59		1.90	—	2.13	2.41	2.38	2.53
8750						1.98	2.20	2.26	2.28
8630	1.64	—	2.81	3.52	—	4.19	5.10	5.46	6.23
8420						4.71	5.07	5.19	5.27
9000-psi series									
9200	1.70	1.94	—	2.27	—	2.51	2.54	2.74	2.73
9230						2.28	2.15	2.48	2.32
11230	2.98	—	3.88	4.21	—	4.64	5.35	5.87	6.22
11650						5.02	5.04	5.78	5.89

shale aggregates from rotary kilns produced the lower-weight concretes. The expanded slag and sintered shale produced the heavier lightweight concretes.

Compressive strength

The tests of structural properties were made on concretes of nominal 28-day compressive strengths of 3000 and 4500 psi for each of the aggregates and at higher strengths for the selected aggregates in the high-strength series. All of the lightweight-aggregate concretes showed about the same strength gain as the sand-and-gravel concrete under similar curing conditions. Most of the concretes drying at 50 percent relative humidity reached a maximum strength at about 90 days, and then showed a small loss of strength. This recession in strength is generally attributed to shrinkage stresses. All strength data are given in Table 3.

Modulus of elasticity

The static modulus of elasticity of the lightweight-aggregate concretes in the 4500-psi series varied from 53 to 82 percent of the modulus of sand-and-gravel concrete, 3.43×10^6 psi, based on values obtained on moist-cured cylinders at 28 days. At 6 months the modulus of the lightweight concretes was 44 to 63 percent of the modulus of 5.01×10^6 psi for the sand-and-gravel concrete. Concretes containing Aggregates 6 and 7 had higher values of elastic modulus than the other lightweight concretes, while concretes containing Aggregates 1, 2, and 4 had the lower moduli. Values of static modulus reported are the secant modulus at $0.3\ f_c'$; almost identical values were obtained at $0.45\ f_c'$. The static moduli of elasticity of the concretes in the 7000-psi series were: Aggregate 9, 3.0×10^6 psi; Aggregate 4, 2.13×10^6 psi; and Aggregate 8, 4.19×10^6 psi. In the 9000-psi series the moduli were somewhat higher.

The ratio n of the modulus of elasticity of steel to that of concrete is compared to values of n established by the ACI Building Code[4] ($30{,}000/f_c'$) in the following table:

Aggregate	28-day compressive strength	E_s/E_c	$30{,}000/f_c'$
Sand and gravel	3000	9.4	10
Sand and gravel	4200	8.8	7.2
Sand and gravel	7000	7.2	4.3
Sand and gravel	9500	6.5	3.2
Lightweights	3000	14 to 20	10
Lightweights	4500	10 to 16	6.7
Lightweights	8500	12	3.5

Values established by two definitions are in fair agreement for the lower-strength sand-and-gravel concretes. However, for high-strength sand-and-gravel concrete and for lightweight concretes there is a large difference between the values established by the two formulas. The relationships be-

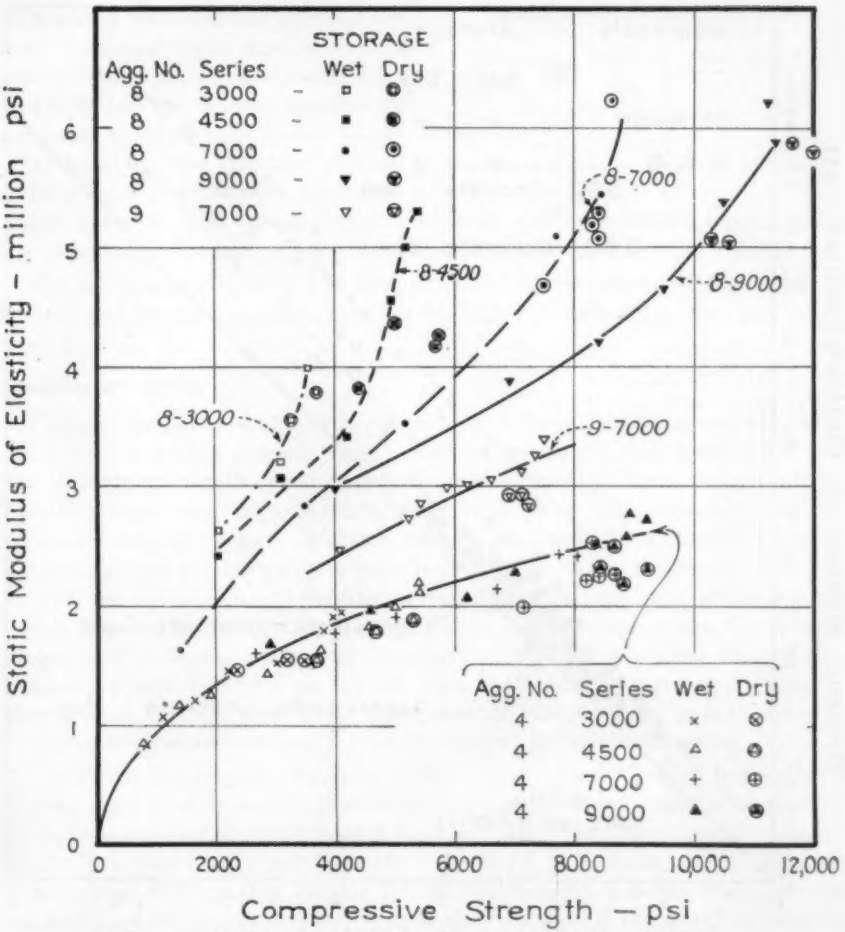

Fig. 4—Relationship between modulus of elasticity and compressive strength

tween compressive strength and modulus of elasticity for some aggregates are plotted in Fig. 4. Four separate curves are required to show the relationship between the modulus of elasticity and the compressive strength of the sand-and-gravel concrete. These curves represent four different concrete mixes and include data to 1 year age. The modulus of elasticity may vary widely for a given compressive strength, depending on the mix proportion and age of specimen. The modulus appears to increase more rapidly than the compressive strength at later ages. This relationship is expressed by a single curve for concrete containing the lightweight Aggregate 4, over a wide range of mix proportions.

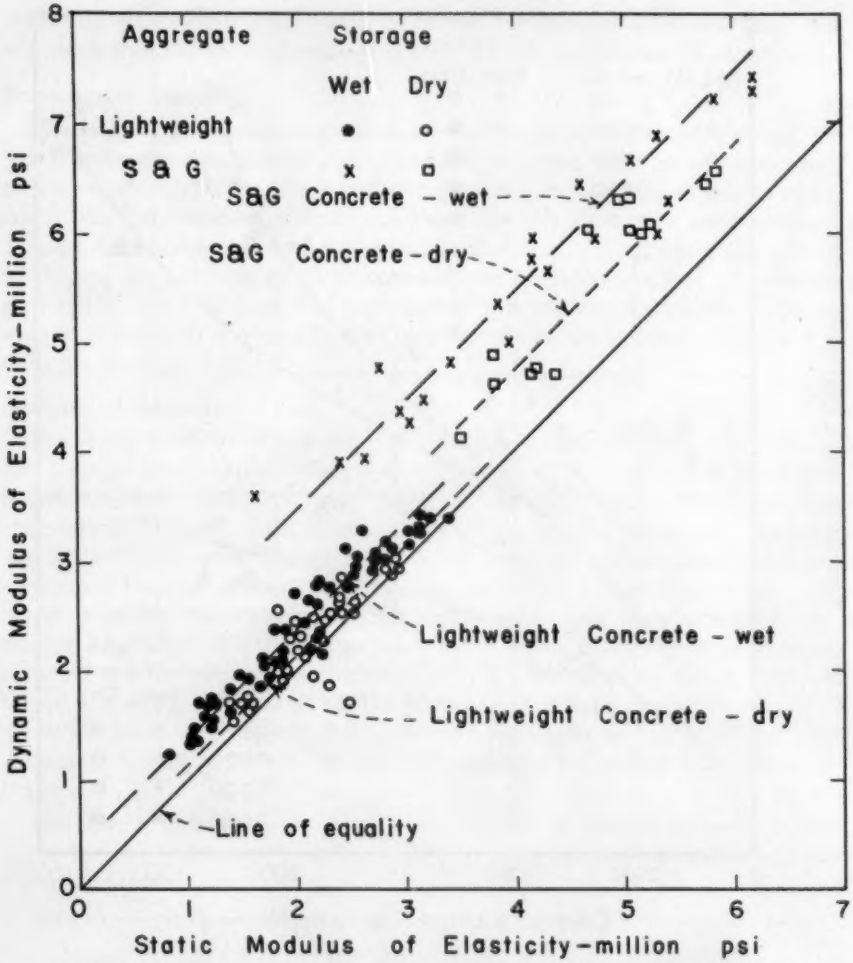

Fig. 5—Relationship between static and dynamic modulus of elasticity

Relationships between static modulus and dynamic modulus of elasticity of both wet and dry concretes are shown in Fig. 5. Values of the dynamic modulus of the moist-cured lightweight concretes were about 350,000 psi greater than the static modulus. As the concretes dried, the difference between the dynamic and static modulus was reduced. The dynamic modulus of sand-and-gravel concrete was about 1,500,000 psi greater than the static modulus for moist-cured specimens and about 750,000 psi greater than the static modulus for dried specimens.

Poisson's ratio

Static values of Poisson's ratio for concretes containing lightweight and normal weight aggregates were approximately equal. Poisson's ratio as

determined by the static method for both the lightweight and sand-and-gravel concretes varied between 0.15 and 0.25 for the various aggregates, ages, and moisture conditions. Values determined by the dynamic method were generally greater for sand-and-gravel concrete than for lightweight concrete, and were greater for wet than dry concrete. Average values from numerous tests are given in Table 4.

TABLE 4—POISSON'S RATIO

Concrete	Condition	Static	Dynamic
Lightweight	wet	0.20	0.22
	dry	0.19	0.18
Sand-and-gravel	wet	0.20	0.23
	dry	0.20	0.21

The low average value of 0.18 was due mainly to low values (0.03) obtained on concrete containing Aggregate 4 in the high-strength series after prolonged drying.

Modulus of rupture

Flexural strengths of the lightweight- and of the sand-and-gravel concretes did not differ greatly at early ages, but after 28 days the lightweights showed less strength gain with continuous moist curing than did the sand-and-gravel. Values of modulus of rupture for the various series are summarized in Table 5. At a compressive strength of about 3000 psi the modulus of rupture for both lightweight and regular concrete was about 450 psi. At a compressive strength of 4500 psi the modulus of rupture of lightweight concrete was about 575 psi and of regular concrete about 640 psi. In the high-strength series, the flexural strength of concrete containing Aggregate 9 was greater than that of the sand-and-gravel concrete at age 28 days, but there was no strength gain thereafter. At later ages flexural strength of the sand-and-gravel concrete exceeded 900 psi and was quite superior to the lightweight concretes.

Lightweight-aggregate concrete beams moist cured 7 days followed by drying at 50 percent relative humidity had a much lower modulus of rupture than the continuously moist-cured beams. Other investigators show similar results for sand-and-gravel concrete. However, the percentage of strength reduction is considerably greater for the lightweight concrete than for the sand-and-gravel concrete. The loss of flexural strength is attributed to internal stresses caused by moisture gradients within the specimens.

The relationship between the compressive strength of concrete cylinders and modified cubes is shown in Fig. 6. The cylinders and cubes had approximately the same strength at about 3000 psi. At 4500 psi the cylinder strengths of the lightweight concretes were about 7 percent higher than the cube strengths. The cube and cylinder strengths of the sand-and-gravel concrete were approximately equal up to 8000 psi, and at higher strengths the cube strengths were slightly lower. The lower strengths shown by the cubes may be due to the fact that the majority of the cubes were not capped. Koenitzer[5] reported that 6-in. modified cubes (capped) and 6 x 12-in. cylinders of normal weight concrete had approximately equal strengths, but that modified cubes not capped were 8 percent lower in strength than the cylinders. Data reported by Kesler[6] show that with normal weight concrete the modified cube speci-

men, capped, gave a compressive strength about 10 percent higher than the standard cylinder for average strength concrete.

Bond strength

High bond strength values were obtained with most of the lightweight-aggregate concretes. Bond stress is plotted against slip at both the loaded and free ends of the bar for each of the aggregates in Fig. 7a and 7b. The bond stress at 0.01-in. slip at the loaded end, used as a criterion of failure, is the basis of the values given in Table 6. A slip of 0.001 in. at the free end is also sometimes used as a criterion of failure; in each case this value indicated a lower bond strength than indicated by slip at the loaded end. The yield point of the steel varied from about 43,000 to 49,000 psi over the entire series of tests.

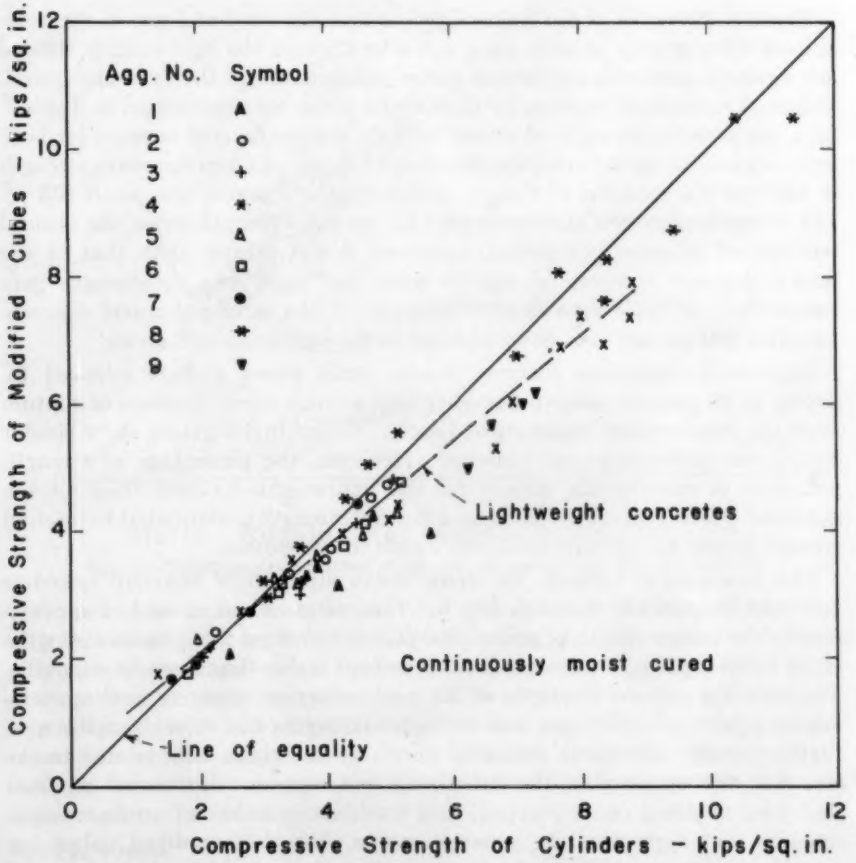

Fig. 6—Relationship between compressive strength of concrete cylinders and modified cubes

TABLE 5—MODULUS OF RUPTURE, PSI (CONTINUOUS MOIST CURING)

Aggregate No.	3000-psi series			4500-psi series			7000-psi series				9000-psi series			
	7 days	28 days	90 days	7 days	28 days	90 days	7 days	28 days	90 days	180 days	7 days	28 days	90 days	180 days
1	345	430	515	415	480	480								
2	360	470	450	530	600	575								
3	385	450	455	510	585	525								
4	290	440	465	430	515	550	520	620	660	665	555	600	645	680
5	385	510	535	490	575	570								
6	365	500	570	515	680	695								
7	295	415	475	435	585	620								
8	335	450	555	500	640	720	635	755	940	945	785	935	1040	1015
9							730	825	770	800				

TABLE 6—BOND STRENGTH, PSI

Aggregate No.	3000-psi series			4500-psi series			7000-psi series			9000-psi series		
	Top bar	Bottom bar	Vertical bar	Top bar	Bottom bar	Vertical bar	Top bar	Bottom bar	Vertical bar	Top bar	Bottom bar	Vertical bar
1	600	840	990	755	970	990*						
2	870	880	900*	900*	900*	900*						
3	670	900*	900*	850	900*	900*						
4	860	895	900*	900*	900*	900*	1530	1480	1550	1490	1480*	1500*
5	610	850	850	770	900	920*						
6	520	650	900*	670	900	960*						
7	580	710	950	640	950	1000*						
8	790	930	950	870	1000*	1000*	1530	1540*	1540*	1550*	1540*	1500*
9							1430	1500*	1550			

*Indicates failure by yielding of steel.

TABLE 7—CREEP COEFFICIENTS FOR SPECIMENS LOADED TO 600 PSI

28-day strength	3000-psi						4500-psi						
Age at loading, days	7		28		540		7		28		540		
Aggregate No.	m_c	n_c	m_c	n_c	m_c	n_c	m_c	n_c	m_c	n_c	m_c	m_c	$\frac{-}{m_c}$, %
1	1.96	40	1.60	55	0.46	52	1.61	44	1.32	81	—	—	—
2	1.58	46	1.38	72	0.37	84	1.35	80	1.12	99	0.47	110	145
3	1.28	86	1.18	88	0.43	146	1.02	80	0.91	111	0.33	91	119
4	1.50	72	1.29	78	0.42	146	1.20	70	1.08	114	0.32	112	135
5	1.45	78	1.30	83	0.40	107	1.04	94	0.95	87	0.32	75	126
6	1.60	67	1.51	88	0.41	85	1.43	87	1.16	78	0.31	80	149
7	2.03	64	1.62	64	0.40	83	1.28	72	1.10	80	0.35	77	157
8	1.28	21	0.99	23	0.23	123	0.91	34	0.74	24	0.15	55	100
Average	1.58	—	1.36	—	0.39	—	1.23	—	1.05	—	0.32	—	—

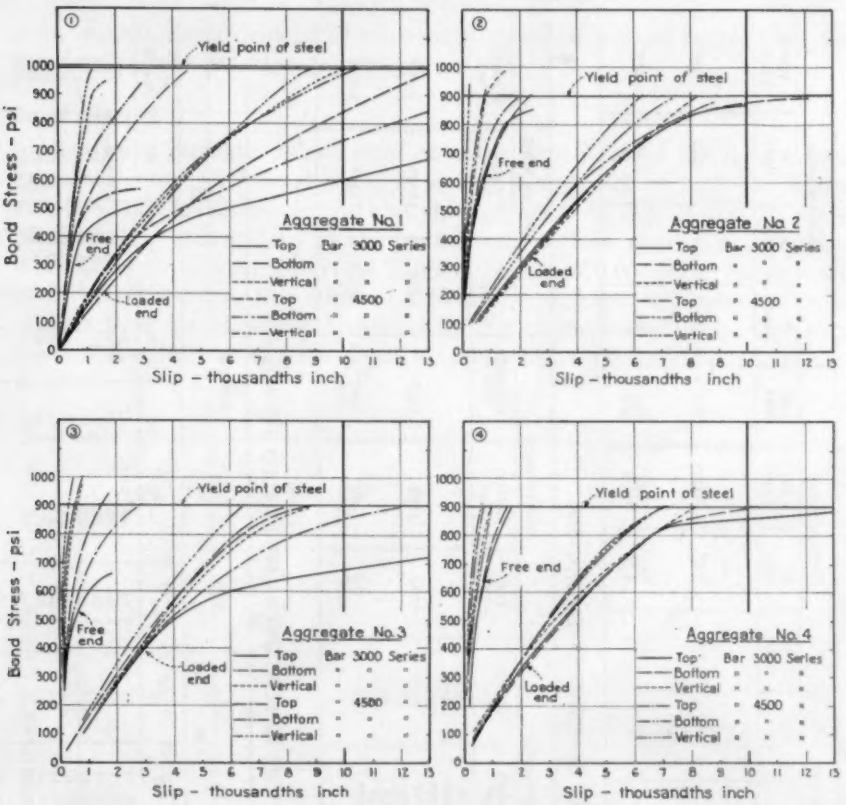

Fig. 7a—Bond strength as affected by strength of concrete, type of aggregate, and position of embedded bar

3000- and 4500-psi series—For the 12-diameter embedment used in the tests the yield strength of the steel was developed at bond stresses of 900 to 1000 psi. In some tests the specimens failed by yielding of the steel (indicated by asterisks, Table 6), and in others the specimens failed by slip at bond stresses in excess of 900 psi.

The position of the bar was an important factor in the development of bond. Top horizontal bars developed the least bond, and vertical bars developed the greatest bond. Low bond strengths for the top bars of concrete containing Aggregates 6 and 7 were undoubtedly due to voids caused by water gain under the bar. These two concretes had a greater tendency to bleed than the others, particularly with the leaner mixes. However, the lowest bond strength obtained, 520 psi, provides a safety factor of 2.5 over the ACI Building Code[4] allowable bond stress of 210 psi for top bars in 3000-psi concrete. Bond and slip performance of the bottom bar only in the various concretes in the 4500-psi series are compared in Fig. 8. The sand-and-gravel

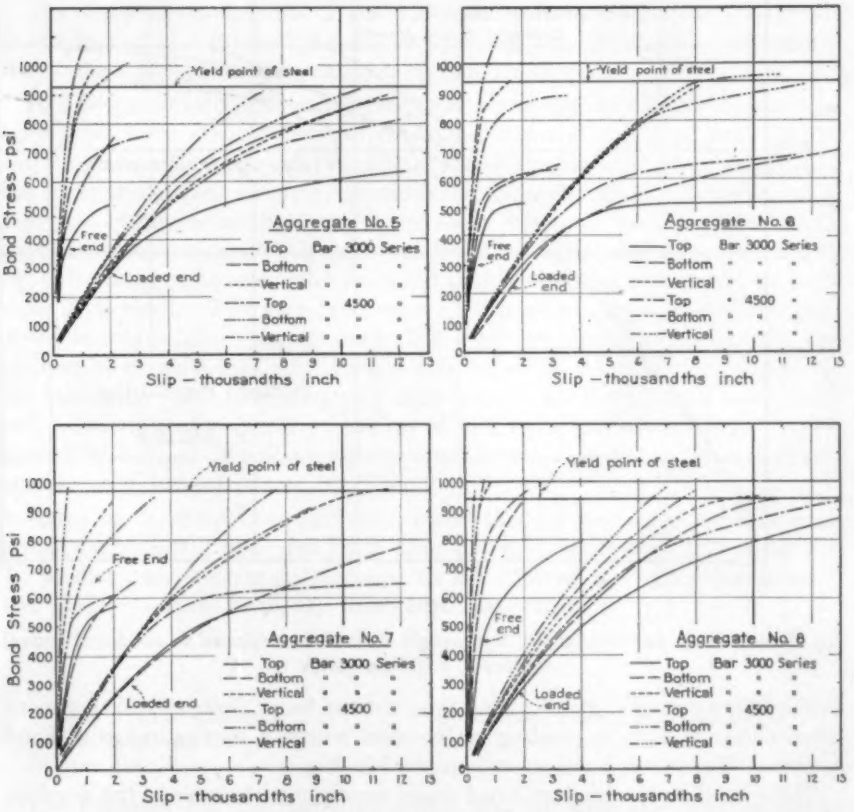

Fig. 7b—Bond strength as affected by strength of concrete, type of aggregate, and position of embedded bar

concrete developed the greatest bond stress, but the margin of superiority was very small since the bottom bar in all of the concretes developed at least 900 psi. Concrete specimens containing Aggregates 2, 3, 4, and 8 failed by yielding of the steel.

Specimens containing the vertical bars in the 4500-psi concrete all failed by yielding of the steel. In the weaker concrete the vertical bar in the sand-and-gravel specimen failed by slip at a bond stress of 950 psi, and all but one of the vertical bars in the lightweight concretes failed at bond stresses in excess of 900 psi.

7000- to 10,000-psi series—High bond strength values were obtained with all concretes. For the eight-diameter embedment of the #9 bars used in these tests, the yield strength of the steel was developed at bond stresses of 1490 to 1560 psi. The lowest bond stress obtained was 1430 psi and many specimens failed by yielding of the steel. The sand-and-gravel concrete had higher bond strengths than the lightweight concretes and the bottom and

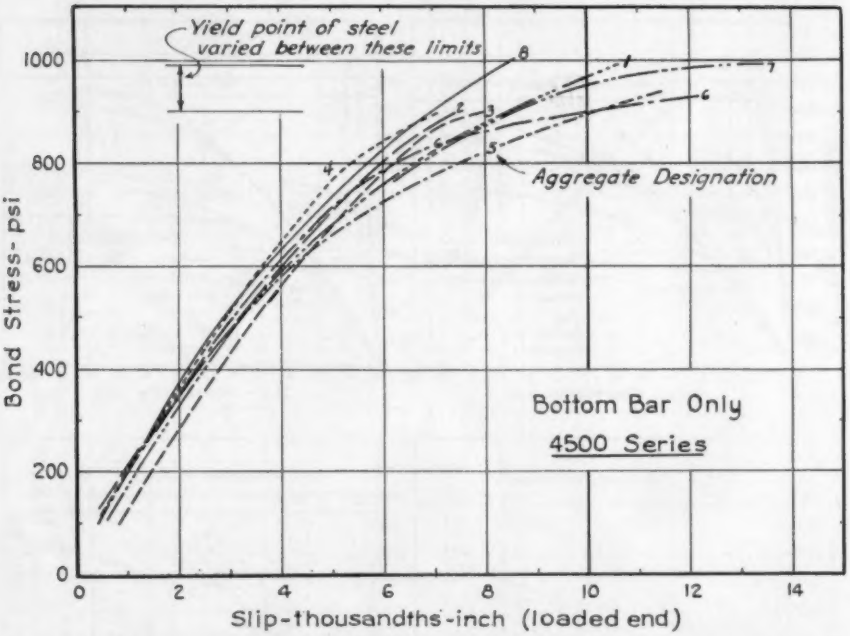

Fig. 8—Bond-slip performance of lightweight concretes compared to sand-and-gravel concrete, 4500-psi series

vertical bars provided greater bond than the top bar. However, all specimens failed either by slip or yielding of the steel within a narrow range of bond stresses. Slip versus bond stress is plotted in Fig. 9.

These bond tests were performed under conditions favorable to the development of high bond strengths, including use of relatively low slump concrete, compaction by vibration, relatively thick section to resist splitting, and the restraining lateral force applied by the testing machine to the face of the specimen. However, these conditions apply to the sand-and-gravel concrete as well as to the lightweight concrete.

Creep

Sustained load tests were made to obtain a comparison between the creep characteristics of concrete containing various lightweight aggregates and concrete containing sand-and-gravel aggregate. Fig. 10 shows the creep strain for 3000-psi concrete loaded at 600 psi continuously after age 7 days. At the time of the 7-day loading, the compressive strengths of the 3000-psi series concretes varied from 1620 to 2270 psi for the various aggregates as indicated in Table 3. Values for creep strain were obtained by subtracting the drying shinkage and elastic strain from the total deformation.

Creep strains of the 3000-psi concretes loaded to 600 psi at age 28 days after 21 days of drying at 50 percent relative humidity were similar to those shown in Fig. 10, but somewhat lower.

The creep characteristics of the 4500-psi concretes loaded at 7 days are shown in Fig. 11. At the time of 7-day loading, the compressive strengths of the 4500-psi series concretes varied from 2940 to 3900 psi.

It has generally been considered that concretes containing lightweight aggregates have greater creep than sand-and-gravel concrete, but Fig. 10 and 11 show that some lightweight-aggregate concretes had less creep than the sand-and-gravel concrete, particularly at early ages. However, at 1 year, six of the seven lightweight-aggregate concretes had developed more creep than the corresponding sand-and-gravel concrete, and the creep of four lightweight-aggregate concretes exceeded that of the sand and gravel by more than 25 percent. The three concretes containing the expanded shales produced in rotary kilns (Aggregates 1, 2, 3) although similar in appearance and method of manufacture, exhibited a wide range in creep values. However, the relatively poor performance of Aggregate 1 is attributed to some flaky and unburned material in the fine sizes of this experimental aggregate. Aggregates 3 and 5 had consistently lower creep strain than the other lightweight aggregates regardless of test condition.

Shape of the creep-time curves indicates that at later ages the sand-and-gravel concrete will have less creep than the lightweight-aggregate concretes. To predict creep values at later ages an equation of the hyperbolic form [7,8] was used:

$$e_c = \frac{m_c\, ts}{n_c + t}$$

e_c = creep strain in millionths
t = time under load, days
s = stress, psi
m_c = ultimate creep, millionths per psi
n_c = time in days when $e_c = m_c/2$

The equation fits the measured points well at later ages, but generally indicates low values of creep at early ages. This type of equation is useful in that it provides a value m_c which indicates the ultimate creep in millionths per psi. The equation was applied to each of the creep strain-time curves for the various test conditions, and values of the creep coefficients m_c and n_c are recorded in Table 7. Some experimental data indicate that drying shrinkage has a pronounced effect on creep, and that because of slower drying larger specimens have less creep than smaller ones. These coefficients then may indicate greater creep than actually will occur in large structural members.

The value $\overline{m}_c$ for each aggregate is the average m_c for the six test conditions (three ages at loading and two compressive strengths) expressed as a percentage of the average m_c for sand-and-gravel concrete. This value of $\overline{m}_c$ is used only as a convenient factor for comparing the lightweight-aggregate and sand-and-gravel concretes and indicates that the ultimate creep of the lightweight concretes will be 19 to 57 percent greater than that of the sand-and-gravel concrete. For increasing values of ultimate creep ($\overline{m}_c$) the aggregates fall in the following order: 8, 3, 5, 4, 2, 6, 7, and 1. The average ultimate creep for the 3000-psi concrete is about 29 percent greater than for the 4500-

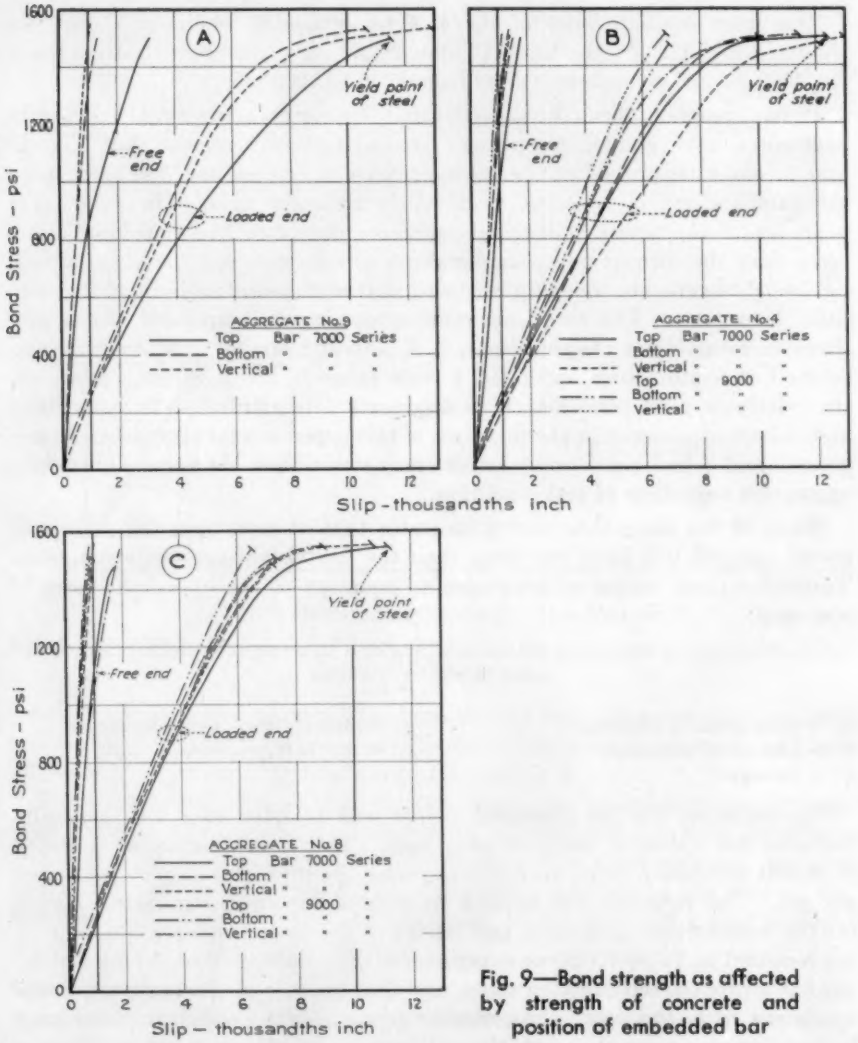

Fig. 9—Bond strength as affected by strength of concrete and position of embedded bar

psi concrete. The average ultimate creep of concrete loaded at 28 and 540 days is 85 and 26 percent, respectively, of the creep of concrete loaded at 7 days. Average creep coefficients for specimens loaded to 2000 psi at 18 months were almost identical to those obtained at the 600-psi load.

Concretes in the high-strength series were characterized by low creep strain. The creep strain curves for 7-day loading are shown in Fig. 12. Concrete creep in the 7000-psi series was 30 to 40 percent of that in the 3000-psi series and about half as much as in the 4500-psi series. The creep coefficients

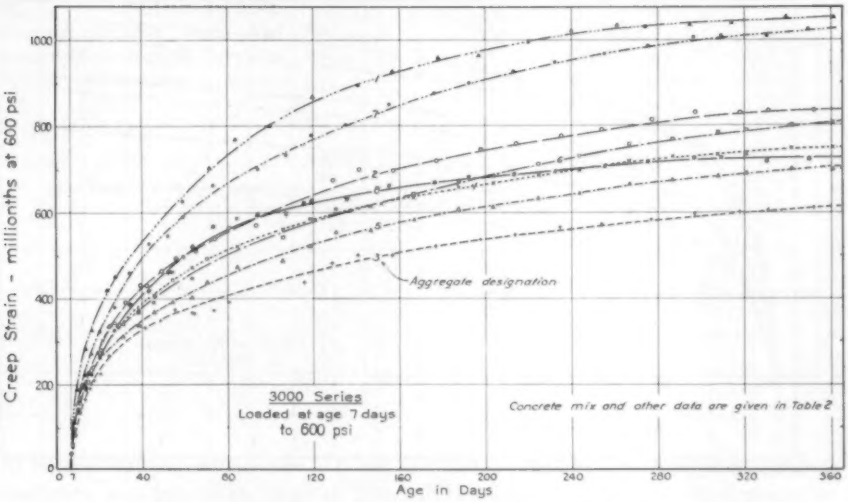
Creep Strain - millionths at 600 psi
Age in Days
Aggregate designation
3000 Series
Loaded at age 7 days
to 600 psi
Concrete mix and other data are given in Table 2

Fig. 10—Creep of concretes containing various aggregates, 3000-psi series

in Table 8 show that concrete containing Aggregate 9 had slightly less creep than sand-and-gravel concrete at the same strength.

Values of ultimate creep of concretes containing Aggregate 4 and concrete containing sand and gravel are compared on a strength basis in Fig. 13. This figure shows that the ultimate creep decreased rapidly as the compressive strength increased to about 6500 psi. At strengths above 6500 psi the reduction in creep was less pronounced. The compressive strength of the concrete at the time of loading is used in Fig. 13.

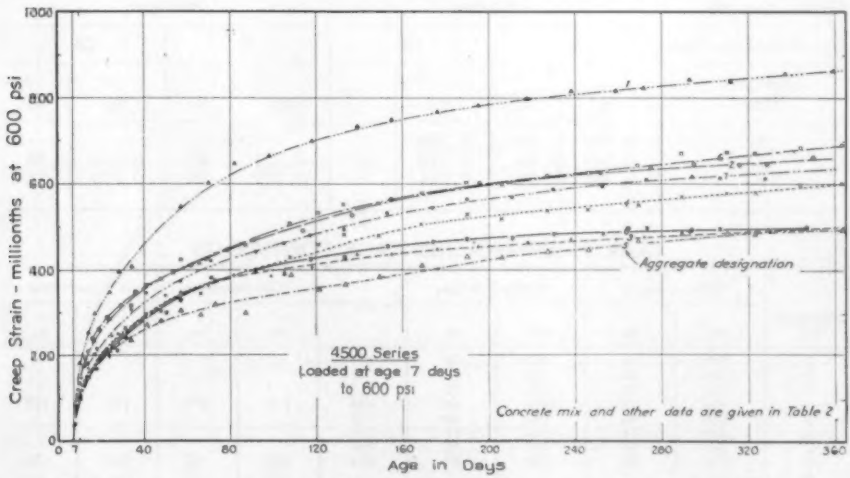
Creep Strain - millionths at 600 psi
Age in Days
Aggregate designation
4500 Series
Loaded at age 7 days
to 600 psi
Concrete mix and other data are given in Table 2

Fig. 11—Creep of concretes containing various aggregates, 4500-psi series

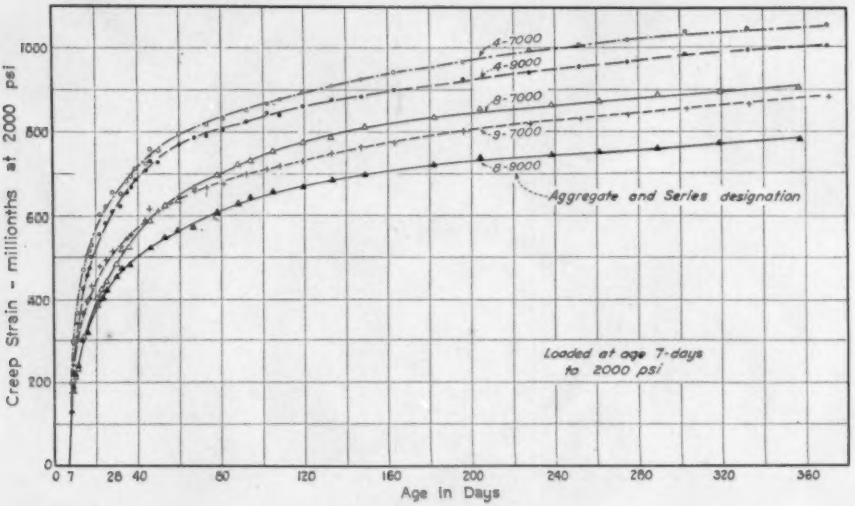

Fig. 12—Creep of concrete containing various aggregates

Drying shrinkage

The drying shrinkage of the lightweight-aggregate concrete was generally greater than that of the sand-and-gravel concrete as indicated in Fig. 14. In the lower strengths the difference was as much as 30 percent at 6 months, and 38 percent at 1 year, although at earlier ages some of the lightweight concretes showed less drying shrinkage than the sand-and-gravel concrete. In

TABLE 8—CREEP COEFFICIENTS—HIGH-STRENGTH SERIES

28-day strength	7000-psi				9000-psi			
Age at loading, days	7		28		7		28	
Aggregate No.	m_c	n_c	m_c	n_c	m_c	n_c	m_c	n_c
9	0.462	29	0.360	39	—	—	—	—
4	0.544	24	0.410	41	0.520	23	0.382	36
8	0.482	28	0.384	35	0.417	28	0.322	34

TABLE 9—DRYING SHRINKAGE COEFFICIENTS

Aggregate No.	3000-psi series		4500-psi series		$\overline{m_s}$, percent	7000-psi series		9000-psi series	
	m_s	n_s	m_s	n_s		m_s	n_s	m_s	n_s
1	885	39	983	34	129				
2	1006	45	987	62	138				
3	885	25	795	67	116				
4	923	31	1066	54	138	769	106	755	173
5	866	25	965	44	127				
6	880	54	940	51	126				
7	745	29	792	41	106				
8	713	20	730	34	100	534	21	620	31
9						476	49	—	—

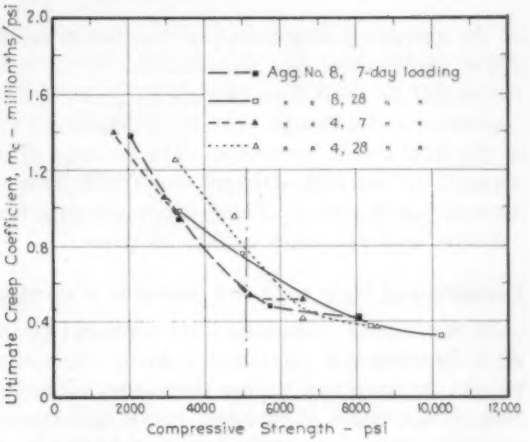

Fig. 13—Relationship between ultimate creep and compressive strength at time of loading

the high-strength series the drying shrinkage of the lightweight-aggregate concretes was less than that of the sand-and-gravel concrete at 6 months. At later ages the shrinkage of the sand-and-gravel concrete was intermediate between that of the two lightweight concretes.

The drying shrinkage-time curves are described by the same type of equation as the creep-time curves, and the ultimate drying shrinkage coefficients, m_s and n_s, are given in Table 9.

The average of the drying shrinkage coefficients in the 4500-psi series was 5 percent greater than for the 3000-psi, but the coefficients did not show consistent variation for the different aggregates. The value $\overline{m}_s$ is the average of the coefficients for the 3000- and 4500-psi series expressed as a percentage

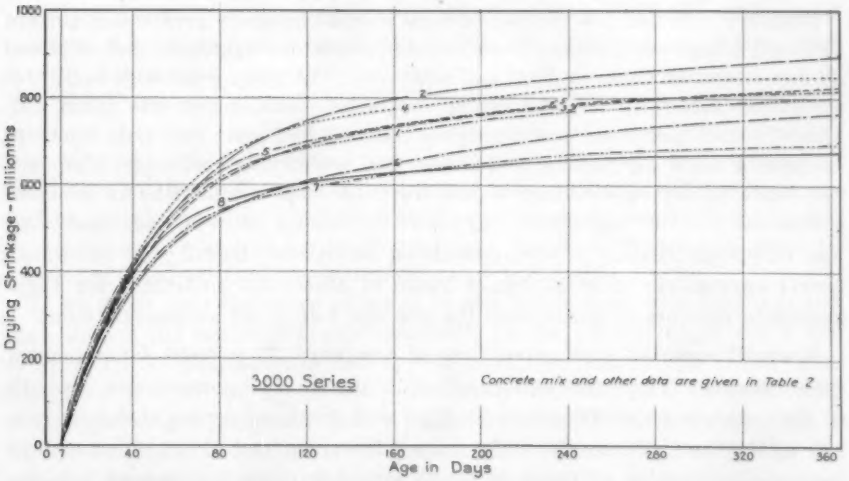

Fig. 14—Drying shrinkage of concrete containing various aggregates, 3000-psi series

of the average drying shrinkage coefficient for the sand-and-gravel concretes. These values show that the ultimate drying shrinkage of the lightweight concretes will be from 6 to 38 percent greater than that of the sand-and-gravel concretes, and concrete containing Aggregate 7 will have the least shrinkage of the lightweight concretes. The average ultimate drying shrinkage of the concrete in the high-strength series will be about 80 percent of that of the lower-strength series. These lower values are due to greater density of the concrete and to higher quality of paste.

Evaluation of Elgin sand and gravel as a standard

Since concrete containing Elgin sand and gravel has been used as a standard, it is important to compare it with other sand-and-gravel concretes with respect to creep and drying shrinkage. Due to the many variables of specimen size, strength of concrete, curing and storage conditions prior to loading, age at loading, and ambient relative humidity during test period, no directly comparable data have been found. Lorman[8] computed the creep coefficients from data by several investigators. These coefficients are subject to the above variables but provide a convenient basis of comparison. The creep coefficients m_c for concretes containing various aggregates in both the fine and coarse sizes were: (from data reported by Davis[9]) sandstone 1.67, basalt 1.38, sand and gravel 1.07, quartz 1.01, and limestone 0.72. The values are for concretes of $w/c = 0.59$, fog cured 28 days, and then loaded and stored at 50 percent relative humidity. The Elgin sand-and-gravel concrete with coefficients of 1.28 and 0.99 for 3000-psi concrete loaded at 7 and 28 days indicate average values. The creep coefficients of the concrete containing gravel aggregate used by Thomas[10] were almost identical to those obtained on Elgin aggregate at the same water-cement ratio.

Schorer[11] reported low values for total length change of prestressed prisms. He used a high-early strength cement and limestone aggregate and obtained 28-day strengths between 5000 and 6000 psi. The creep coefficient m_c for his specimens was about 0.5 and drying shrinkage coefficient m_c was about 350. Davis[9] found that limestone aggregate concrete had less creep than concrete containing other aggregates, and it also had low drying shrinkage. Carlson[12] also reported drying shrinkage values from 350 to 870 millionths for concrete containing various aggregates. Concrete containing limestone aggregate had less shrinkage than concretes containing sandstone, basalt, and sand-and-gravel aggregates. The shrinkage value of about 700 millionths for Elgin aggregate concrete is higher than the average, but is not an unusual value.

Koebel[13] reported steel stress loss of less than 25 percent for expanded shale concrete beams post-tensioned at 32 days. The compressive strength of the concrete was 6000 psi at 28 days and the total drying shrinkage was 720 millionths in 6 months. His values for creep and shrinkage would be approximately equal to those obtained with Aggregate 3 if corrected to the same load.

Alkali-aggregate reaction

The particular aggregates under study have shown no evidence of alkali-aggregate reaction after 9 months storage over water. Alkali content of the cement was 0.63 percent expressed as Na_2O.

SUMMARY

1. Structural grade concrete was obtained with each of the lightweight aggregates.

2. The unit weights of the various lightweight-aggregate concretes in the lower strength series ranged from 90 to 110 lb per cu ft compared to 146 for sand-and-gravel concrete. The expanded shale aggregates from rotary kilns produced the lower-weight concretes. The expanded slag and sintered shale produced the heavier lightweight concretes.

3. The various lightweight aggregates require a wide range of cement content to produce similar strengths. The 3000-psi concretes required between 4.4 and 7.2 bags per cu yd and the 4500-psi concretes required between 6.0 and 8.9 bags per cu yd. Strengths in excess of 8000 psi were obtained with one of the lightweight aggregates in a 10-bag mix. The strength gain of the lightweight concretes under continuous moist curing and under drying at 50 percent relative humidity was similar to that of sand-and-gravel concrete.

4. The modulus of elasticity of the lightweight-aggregate concretes in the 3000- and 4500-psi series varied from 53 to 82 percent of the modulus of sand-and-gravel concrete, based on values obtained from moist-cured cylinders at 28 days. At later ages there was a greater difference between the moduli of the lightweight and normal weight concretes.

5. The flexural strengths of the lightweight and sand-and-gravel concretes were approximately equal at early ages, but after 28 days the sand-and-gravel concrete showed greater strength gain with continuous moist curing than did the lightweight concrete.

6. Bond strengths of some of the lightweight concretes were approximately equal to those of the sand-and-gravel concretes. Lowest bond strengths were obtained with top-position bars in concretes which showed pronounced bleeding. In the 3000- and 4500-psi series, all specimens, with a single exception, failed either by yielding of steel or at bond stresses in excess of 900 psi when the bar was in a vertical position. In the high-strength series all specimens failed at bond stresses in excess of 1430 psi.

7. At early ages the creep of some of the lightweight concretes was less than that of the sand-and-gravel concrete. In the lower-strength series two of these lightweight concretes had approximately the same creep as the sand-and-gravel concrete at age 1 year. However, the computed ultimate creep of the lightweight concretes, with a single exception, is greater than that of the sand-and-gravel concrete, as indicated by the creep coefficients.

8. In the lower-strength series at age 6 months the drying shrinkage of the lightweight concretes stored at 50 percent relative humidity was between

95 and 138 percent of that of the sand-and-gravel concrete. The predicted ultimate drying shrinkage of the lightweight concretes for 50 percent relative humidity is between 106 and 138 percent of that for sand-and-gravel concrete as indicated by the average coefficient $\overline{m}_s$. In the high-strength series the drying shrinkage of the sand-and-gravel concrete was intermediate between the two lightweight concretes.

9. There was no evidence of alkali-aggregate reaction with any of these aggregates when tested with a cement having an alkali content of 0.63 percent.

10. Within the group of lightweight aggregates studied rather wide variations were obtained in the structural properties of the concretes. This was true of lightweight aggregates that were similar in appearance and produced by similar processes. It is important, therefore, that individual producers of lightweight aggregates for structural concrete purposes conduct investigations that will provide reliable design data on performance characteristics.

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Title No. 54-17

Construction of the Dallas Memorial Auditorium*

By JACK E. ROSENLUND†

SYNOPSIS

Main structure of the Dallas Memorial Auditorium has 204 ft diameter, doubly curved roof cast of lightweight concrete in 16 separate pie-shaped sections; roof is supported from 45-ft rigid frames that cantilever from 70-ft columns arranged at the periphery of the auditorium, providing a 300 ft diameter clear area. Paper recounts details and problems of forming the columns, cantilevers, and roof sections as well as general considerations governing their design. Problem of cambering forms to compensate for anticipated deflection is treated, along with proportioning, mixing, and placing of both the lightweight and normal weight concrete.

ARCHITECTURAL DESIGN CONSIDERATIONS

The city of Dallas required an all-purpose building that could be used for conventions, operas, circuses, sports events, and various exhibitions, with ample parking space and facilities for public transportation. Facilities were also needed for trucking in equipment for exhibitions and conventions. By nestling the building into a rolling site, entry was provided into all three working levels from ground level. The project is really two buildings joined together by a passageway, but this paper focuses on the larger, more unusual structure.

Simple geometry showed that a circular building enclosed the most area with the least perimeter, and a curved roof resulted in a saving of space while not limiting the accommodations and usefulness of the building. Since the

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building was to be completely air conditioned, smaller volume would result in more economical operating costs.

Important architectural features consist of highly efficient acoustical treatment, a varied and intricate lighting system which is completely adaptable and interchangeable for the different events, permanent facilities for radio and television broadcasts, and complete installation of utilities for any type of convention or exhibition.

A structure of the Dallas Memorial Auditorium's type, shape, and size presented peculiar problems in design and construction, and of necessity introduced certain ideas and developments novel to long-span construction. Among its unusual design features was the division of a 204 ft diameter, doubly curved roof into 16 separate pie-shaped sections which were supported from 45-ft rigid frames, cantilevering from 70 ft high columns. This provided a 300 ft diameter clear area (Fig. 1).

An equally important consideration was economy. Concentrated study of the use of materials, preliminary designs, and cost estimates preceded the over-all design and material selection. An earnest effort was made to keep the project and its component parts, even to the most minute details, as simple as possible; this, we believe, was instrumental in receiving some surprisingly low bids from the contractors.

STRUCTURAL CONSIDERATIONS AND DESIGN

After a circular-shaped layout was selected as most suitable for the varied uses of the building, structural schemes considered were clear span thin shell concrete, structural steel framed dome, arches of concrete and steel, and precast concrete sections. Reinforced concrete was selected as the structural material because estimates indicated it was the most economical, it afforded minimum maintenance and fireproof construction, and offered a heavier roof mass, which is desirable for acoustical purposes.

In the early stages of design, the architects and engineers considered a clear span dome of some 300 ft with the roof supported on columns on the outside perimeter of the building. However, it was found advantageous to separate the roof into arched, pie-shaped sections in order to allow for reusable forms, thereby reducing an important cost item. This separation of the roof construction permitted casting of the roof in smaller sections in opposing pairs, which were completely stable while the forms were moved from one location to another (Fig. 2). The total amount of concrete and reinforcing steel was not appreciably increased over that needed for a full dome, but form expenses were so reduced that total cost was about half of that estimated for either a full concrete dome or a structural steel dome.

The pie shapes which comprised the roof were completely separated, except for a 22 ft diameter solid concrete plate at the top of the roof, which was cast with the first roof sections. Stresses due to deflection and volumetric changes were prevented by the separations in the structural system. The

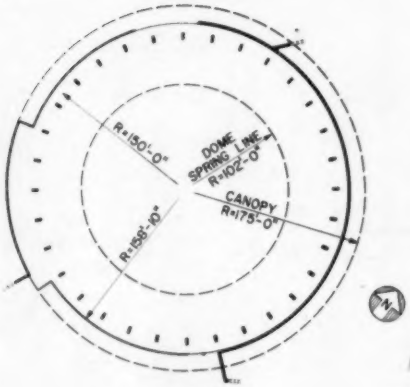

Fig. 1—Auditorium plan

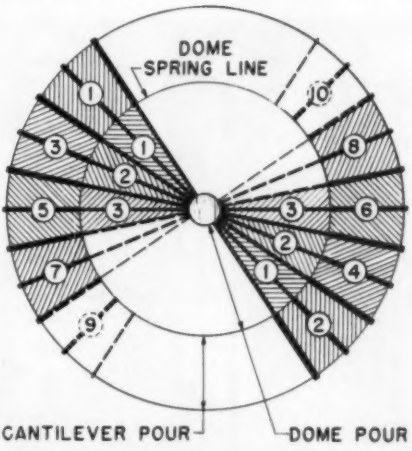

Fig. 2—Diagram of planned concreting

seating slabs which were supported on the outside wall from brackets on the verticals in the cantilever frames, rested on huge sliding bronze bearing plates in order to prevent excessive stresses due to deflections of the roof.

Column size in the cantilever frames was determined by bending stresses due to the cantilever and the load and outward thrust of the arch ribs. An optimum size was reached after various studies of the span of the arch ribs. By reducing the span of the arch ribs, the thrust was reduced, and at the same time the stresses due to the vertical reaction of the arch ribs in the cantilevers were balanced by the outward thrust of the arch ribs on the cantilever frames. A typical bent is shown in Fig. 3. The hanging of mechanical equipment and press rooms under the cantilever frames further reduced the stresses in the cantilever frames due to the thrust of the arch ribs.

The cost of the balcony seating areas was reduced by designing a one-way radial slab spanning between concrete beams, running peripherally so as to take advantage of reusable and movable forms. Although this meant a heavier type of construction, the savings in form costs more than offset the additional material costs.

The building is founded on drilled piers anchored into bedrock. The first floor is a two-way flat slab supported on bell top columns. Verticals of the cantilever frames are tied at the first floor level, and the first floor is designed to carry the combined load of bending due to floor loads, plus axial tension due to roof thrust. The exterior basement walls are essentially retaining walls designed for earth embankments 28 ft high in one case.

FORMING CONSIDERATIONS AND FORMS ADOPTED

During the foundation stages of the project, approval was obtained for special formwork required for the upper portion of the structure. The contractor scheduled first floor concreting before the basement in order to erect

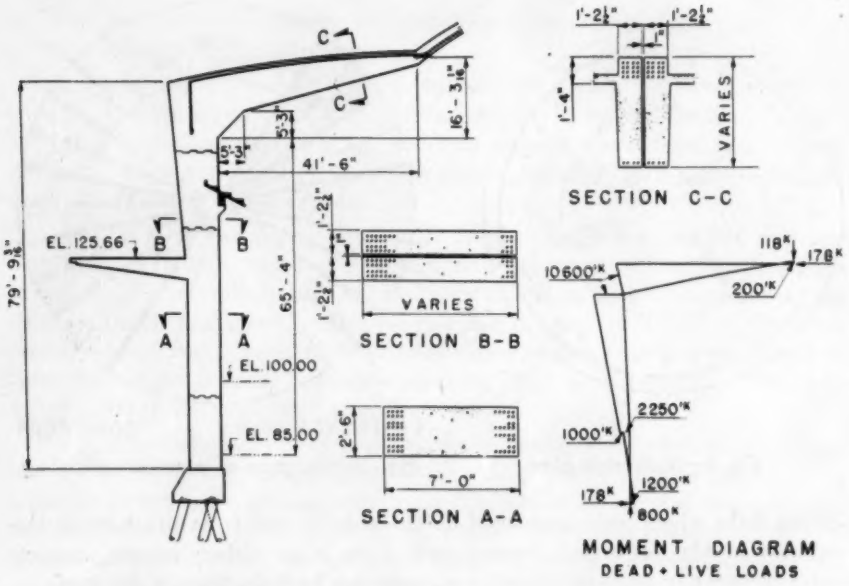

Fig. 3—Typical cantilever bent and moment diagram

the traveling forms for the roof structure. The first floor was formed with a plywood deck, 4-in. joist and girder members supported on 4- x 4-in. shores, 17 ft high, which rested on 2- x 12-in. mud sills.

Columns

Steel was selected for the forms of the 7- x 2½-ft columns of the cantilever frames, since the forms originally were to be used a minimum of eight times; actually some sections were used 32 times. Steel proved to be easy to set and adjust to the intricate camber settings required.

Steel channels and angles were used for the frame of the forms while 3/16-in. plate formed the facing. Three separate sections were required to form the varying shape of the cantilever frame columns. The first section was a length from the footing base to the first floor with blockouts for dowels to receive beams for ramps and slabs which were to be cast later. The second section (Fig. 4) was constructed to include a 25 ft long cantilevered canopy beam whose outside shoring was to remain in place until the forms for the main cantilever frame were removed. Height of the second section was 31 ft 4 in. above the first floor, and it contained #11 bars that extended 32 ft above the top of the form. Placing of the reinforcement required exact positioning; the contractor found that prefabricating sections on the ground was an economical measure. After fabrication, whole sections of reinforcing steel were lifted into place and guyed while permanent supports were placed. This made the reinforcing steel section a self-supporting member.

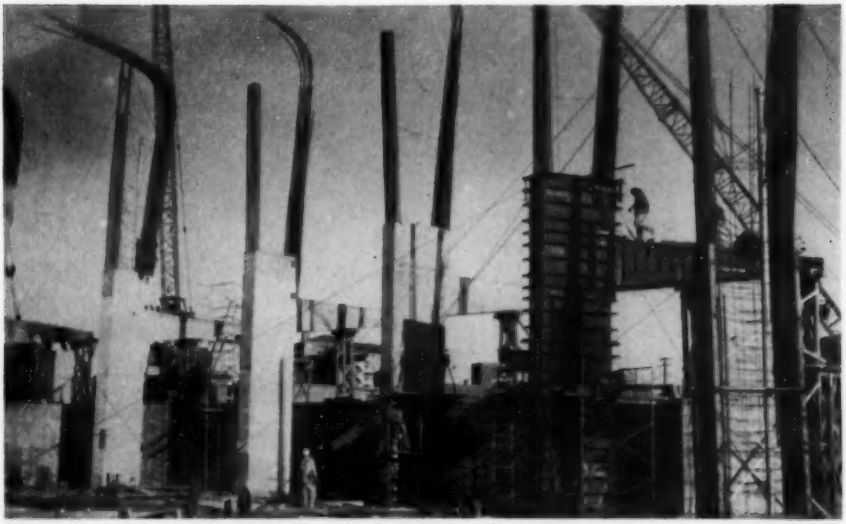

Fig. 4—Second section of column form, in place at right, includes cantilevered beam for auditorium's outside canopy. Shoring for this beam remained in place after forms were stripped, until main cantilever was completed at top of column

The third form section of the column part of the cantilever frame was the topmost vertical form. Starting at the bottom of this form, alternate columns were split from a 2 ft 6 in. wide column to two columns 1 ft 2½ in. (Fig. 5). This split separated the cantilever frames and continued to the top of the roof as dividing line between the pie-shaped sections. A special problem confronted the contractor in forming the 1-in. split. He solved the problem by using steel plates separated with tempered steel rods which were graduated from ⅜ in. to ⅞ in., depending on the thickness of the split. The split was formed in a wedge shape to compensate for cambers and deflections due to the weight of the roof. Screw jacks were used to remove the rods from between the plates and then the plates were lifted out by crane (Fig. 6).

Cantilever section

Each cantilever form was made in two traveling units which were assembled by bolting into position, forming a full cantilever rib in the center and one split rib on each of the two outside edges of the form (Fig. 6). The form for the cantilever frame supported 128 cu yd of concrete on 4465 sq ft 65 ft above the first floor level. To prevent damage to the floor, the contractor designed special timber shoring to support the concentrated wheel loads of 72,000 lb. Special timber trestles were constructed to carry wheel loads over slab openings and ramp openings. In the decentering and moving operation of the cantilever frame form, the traveler moved downward some 8 ft in order to be moved in toward the center of the building (Fig. 7). The depth of the cantilever frame and also the convergence toward the center of all the frames

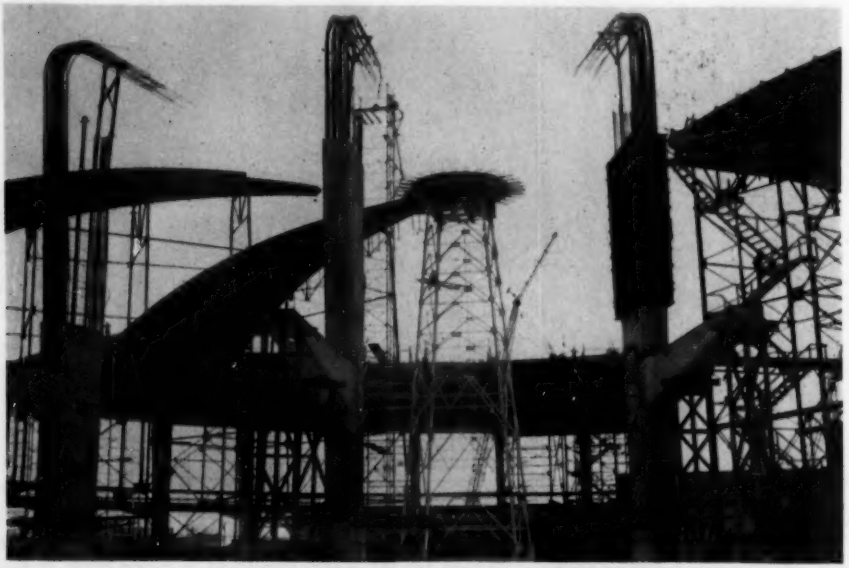

Fig. 5—Form in place for third section of column on right. Note split in adjacent column where form has already been stripped. View also shows first forms in place for roof section

made this movement necessary. After dropping the form 8 ft, it was moved toward the center 13 ft and then rolled to its new position before being raised again.

The form for the cantilever frame was so constructed that while the soffit of the form supported the load, the beam sides could be unbolted and released. This allowed a saving in forming time. The decentering of the cantilever form utilized the following operation: screw jacks lowered the form 4 in. The weight was then transferred to a hydraulic system which lowered the forms 8 ft after the screw jacks were removed.

All roof forms had flanged wheels rolling on rails. A full day was required of the form crews for the preparation of the cantilever form move; decentering, moving, and setting in the new position required another day.

Cambering

Since the roof was designed to be cast in separate sections, it was necessary to compensate for anticipated deflections by cambering the forms. The most intricate cambering was necessary in setting the forms for the cantilever frames. In design of the roof, construction stresses and steel areas were computed by conventional methods for various loading conditions using an elastic modulus of 2,000,000 psi for the concrete. To determine the effect of long-time loading or creep (dead load only) in order to calculate deflections and determine cambers, the elastic modulus was reduced to 750,000 psi. The 750,000 psi is based on long-time-test load data. Although the assumed

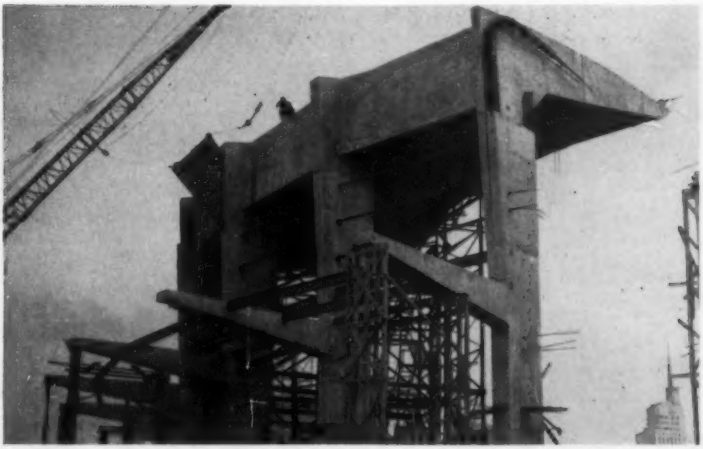

Fig. 6—Removal by crane of plate which formed split in cantilever frame. Full cantilever rib at center, split rib on right

stiffness of the concrete alone is reduced considerably by the low modulus, the total assumed stiffness of the transformed section is not affected very much because of the increased effectiveness of the ideally distributed reinforcing steel with its constant elastic modulus. Therefore, the assumed total change in stiffness due to creep is only about 25 percent.

These effects were considered in the deflections due to both bending and rib shortening to determine final deflections and cambers of the roof. Vertical cambering was required because of the dead load of the cantilever, cambering of adjacent cantilevered sections above previously cast sections, and because of the load of the arched roof resting on the end of the cantilever. Horizontal cambering was required because of the thrust of the arched roof.

In order to compensate for the setting of cambered forms against previously cast sections, the forms were equipped with shim plates for accurate setting. The traveling frame supporting the forms was too rigid to be warped for the different cambers necessary to compensate for anticipated deflection, but the upper forming surface was easily warped.

Roof

The roof cap was designed to be placed before or with the first opposing set of arch pie-shaped units. This cap form had to carry the greater portion of the entire roof until the roof was completely covered. Unbalanced thrust action from concrete and reinforcement placement also had to be considered in the design of the form for the roof cap. Weather conditions and the hanging of structural steel under the cantilever frames were also considered. When the contractor designed the supporting form for the cap, considering both vertical and horizontal loads, his design so resembled an oil derrick that he investigated the possibility of using such a tower. An oil derrick proved

Fig. 7—Several completed cantilevers; form and shoring still in place at right

economical and was erected in the center of the arena over temporary concrete columns extending into rock for support. The derrick was fitted with sand jacks for lowering when concreting of the roof was completed. The roof cap was then cast with dowels extended and construction keys to receive the arch sections (Fig. 8).

The steel form for the arch sections was constructed in two parts for casting opposing sections of the roof simultaneously, thereby maintaining balance while forms were being moved. The decentering of the form had to be uniform to maintain equal forces on the cantilever frames which support one end of the arch section. The moving of the forms merited special attention because of the wind and motion on top-heavy forms. Forms for the arch sections (Fig. 5, 9) were moved in much the same manner as the cantilever frames.

When the arch sections were approximately 75 percent complete, the contractor began erecting structural steel under the roof to support mechanical, electrical, and other essential items necessary for a building of this type. A special crow's nest at the end of a jib boom on a crane was used by erectors who made bolted connections for the framing under the roof.

Decentering of the roof was accomplished before the structural steel hanging under it was complete. By removing sand simultaneously from each of the sand jacks, the roof was gradually decentered. After 6 hr of slow removal of sand from the jacks, the form finally broke loose from the roof concrete where the vertical deflection measured 3⅜ in. After 3 months, the deflection was only 3¾ in.

Miscellaneous forming

After the traveling roof forms were dismantled and cleared from inside the building, erection of forms for the loge seating, balcony seating, and second floor proceeded. These traveling forms were of wood with many re-uses of each form section. The beams supporting the balcony slab were designed to run peripherally so that no beams would interfere with the moving formwork. One-way slabs spanned between beams. Because of the spans and the shape of the sections the balcony slab soffit was formed like an inverted cone, being thicker at midspan than at the supports. Specially designed riser supports were used to hold the balcony risers to line and grade.

CONCRETE PROPORTIONING AND MIXING

Standard weight concrete was mixed from crushed stone, fine aggregate, and cement. Lightweight structural concrete was mixed from cement and coarse and fine lightweight expanded shale aggregate. The coarse lightweight aggregate varied in size from ¾ in. to No. 4, and lightweight fines varied in size from No. 4 to No. 100. It was found that the substitution of approximately 25 percent of standard weight sand for lightweight fines greatly improved the workability and strength, while only increasing the weight by 7 to 10 percent. Lightweight structural concrete was used for all the roof construction above the vertical leg of the rigid bents supporting the roof, plus the "Cofar" utility slabs suspended from the roof.

Specifications called for a minimum of 3750 psi for all concrete, except slabs on ground. Minimum cement specified was 7 bags per cu yd for the lightweight concrete or 6½ bags for the standard weight concrete. Maximum water specified was 5 gal. per bag and slump specified was 1½-4 in. for the lightweight concrete, and 3-5 in. for standard weight concrete.

In every case concrete strength exceeded that specified with, 7-day strengths exceeding that specified in approximately 80 percent of the cases. It was not unusual to receive 28-day test reports of concrete cylinder breaks in excess of 6000 psi for standard concrete and 5500 psi for lightweight concrete.

The contract specified a minimum time after concrete placing for form removal. This was modified by a minimum strength table. In all cases the

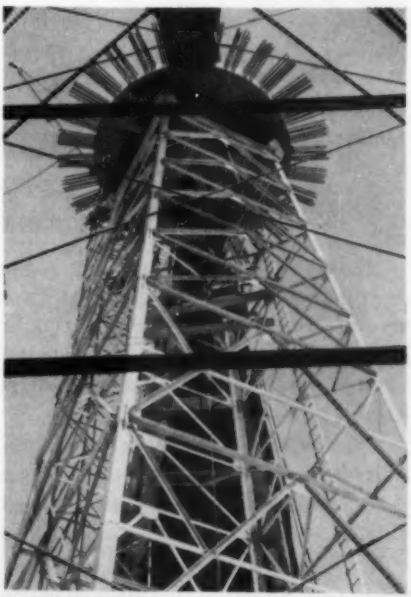

Fig. 8—Oil derrick resting on temporary concrete columns supported roof cap during concreting of entire roof

Fig. 9—Two opposing steel forms for roof sections

contractor found it advantageous to make strip cylinders which cut his waiting period considerably. The forms for the entire roof construction could only be removed after a minimum strength of 2500 psi and elastic modulus of 2,000,000 psi were reached. The elastic modulus was obtained by the sonic method and often exceeded 3,000,000 psi. The average elastic modulus was about 2,500,000 psi.

Since the auditorium is in the heart of downtown Dallas, the contractor set up a concrete batching and mixing plant on the site for better control of the mix and to eliminate haul problems of transit mix. The concrete plant handled coarse and fine expanded shale, sand, gravel and cement. The contractor built a railroad siding to a nearby railway line to bring in concrete batching materials. Giant trailer dumps hauled in materials when street traffic permitted.

Stockpiled materials were fed into a single hopper elevator by a front-end loader. The batching plant operator controlled the gates of the hopper elevator to chute material into the proper bins.

The concrete plant turned out a maximum of 300 cu yd daily. It batched approximately 30,500 cu yd on the job, of which some 3000 cu yd was lightweight structural concrete.

PLACING CONCRETE

Since haul distances did not exceed 1000 feet anywhere on the job, four Dumpcrete trucks proved ideal for concrete hauling. The distance was too short to warrant concrete agitation in truck mixers, but the distance was long enough and the terrain poor enough to rule out hand carts and powered buggies except for placing the roof, where concrete was hauled in hand carts from a central hopper.

The trucks fed concrete buckets swung by long-boom cranes. Almost everything was too high or too heavy to be hauled by ramps or towers. Scaffolding and hoisting towers supported and raised men and lightweight materials only.

In steel form units for the vertical columns, the closeness of the reinforcement and distances between construction joints necessitated gates for placing concrete in the sides of the forms. In some cases, placing was facilitated by lowering a man into the forms with a vibrator.

All of the concrete on the outside of the building and under the roof was placed using cranes with a bucket mounted at the end of the crane boom. The central 22 ft diameter plate at the top of the roof was cast using a construction tower and a hopper under the tower. This plate became a working platform for supporting the rolling steel forms for the 200-ft arch roof. This roof was divided into 16 pie-shaped sections and the sections were cast oppositely for stability so that the form could be lowered and moved into new positions, raised to the proper elevation, and the concrete again placed. Concrete for the arch sections was raised on the construction tower to the working platform mentioned above and deposited in buggies. The buggies were moved out over runways supported off the rolling steel forms and the concrete was dumped into a hopper and chute system which distributed it evenly across the steel forms.

The last pie-shaped section of the roof was placed using a crane placed on the outside of the building hoisting concrete to platforms built over the rolling steel forms. Here the concrete was put in a bucket and chute system which discharged it over the forms. Placing concrete in each of the pie sections took place in a single day and required little more than 8 to 10 hr depositing time.

The largest single problem on this project was coordination. The design of the building was such that sequence work was essential. In conventional construction, one operation may often precede another, or can be so scheduled as to work in conjunction with another, but on the Dallas Memorial Auditorium, each operation had to fall into place before the next could proceed, like pieces of a giant jig-saw puzzle. The success of such a project requires the coordination of the architect, the engineer, and the contractors.

ACKNOWLEDGMENTS

George L. Dahl, Architects and Engineers, Dallas, prepared the architectural design for the Dallas Memorial Auditorium; Ammann and Whitney, New York and Milwaukee, were consulting engineers who prepared the structural plans. R. P. Farnsworth & Co., New Orleans, was general contractor, and steel forms were constructed by Dixie Form and Steel Co., San Antonio.

Phillip Vortisch and Joe Sullivan were project superintendent and project engineer, respectively, under the direction of R. P. Farnsworth, Jr., for the contractor. Boyd Anderson and Allan Anderson represented the consulting engineers in field supervision, and Jack Ratcliff and David Walker were on the job to supervise and coordinate on behalf of the architect.

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Title No. 54-18

Ultimate Torsional Properties of Rectangular Reinforced Concrete Beams*

By G. C. ERNST†

SYNOPSIS

Principal object of this investigation was to determine the quantity of transverse steel required to develop the yield point in longitudinal bars placed in the corners of rectangular beams subjected to pure torsion. Eighteen tests are reported: six each with #3, #4, or #5 longitudinal bars, one in each corner. Transverse ties were spaced at 28, 14, 7, and 4 in., and also in pairs at 4 in. for each group, all of #2 size. One beam in each group contained no transverse reinforcement, and the nominal concrete strength of all groups was 4000 psi.

Results indicate that yield strains can be developed in longitudinal corner bars as well as in transverse ties, resulting in either a diagonal tension type of fracture or a hybrid failure in transverse shear and diagonal tension. Initial cracking corresponded to the failure of unreinforced concrete in torsion for all beams, at an average unit shearing stress of 312 psi. Evidence also developed supporting the concept of a transition from elastic to plastic states of stress as the ratio of transverse to longitudinal steel approaches unity.

INTRODUCTION

Object and scope

Torsional moments in reinforced concrete structures are generally of a secondary nature, but structural applications also occur in which the primary stresses include those resulting from torsion induced by unsymmetrical loading. In the latter case it is necessary to know how to proportion and space the longitudinal and transverse steel, if the structure is to function properly. Previous investigations (see references) have not been directed specifically toward a systematic identification of the interaction between the concrete and the longitudinal and transverse steel at ultimate load. It is at ultimate capacity that a structural member exhibits its energy-absorbing properties under extremes of loading. It is the object of this investigation to determine the optimum amounts of longitudinal and transverse steel for rectangular reinforced concrete members in pure torsion.

The investigation is limited to pure torsion, with three longitudinal steel ratios, six transverse steel ratios, and one concrete strength. Strain measurements on the steel were taken at appropriate locations for the determination

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of yielding in the transverse and longitudinal steel, if and when it occurred. The steel ratios were selected and combined with the intent of demonstrating the development of longitudinal steel yield by means of transverse ties, and also the development of the shearing strength of the concrete when diagonal tension failure is inhibited.

Notation

The letter symbols used in this paper are defined as follows:

T = torque, in.-lb. T' denotes upper limit of torque based on concrete shearing strength
e, p, c, s = subscripts denoting elastic, plastic, concrete, and steel, respectively
b = width of rectangular section, in.
b' = width of reinforcing cage, in.
t = depth of rectangular section, in.
t' = depth of reinforcing cage, in.
v = unit shearing stress, psi
v_c' = transverse shearing strength of concrete, psi
k = coefficient for torque equations, with appropriate subscripts as defined above
k_v, k_h = coefficients for b and h, respectively, to obtain torque arm of steel force at plastic ultimate torque
ϕ = detrusion, radians per in.
θ = total angle of twist, radians
G = modulus of rigidity of concrete
A_L = total cross-sectional area of longitudinal steel, sq in., placed symmetrically around the periphery of the reinforcement cage
A_v = cross-sectional area of one bar of the diagonal tension steel, sq in.
C = plastic detrusion coefficient
f_v = tensile stress in diagonal tension reinforcement, psi
s = spacing of vertical ties, in.
T_c = torque at which initial concrete cracking occurs, in.-lb
Y_L = yield of the longitudinal steel (on charts)
Y_T = yield of the transverse steel (on charts)
n = E_s/E_c
α, β, λ = constants of the elastic theory of torsion
L = total length of beam, ft or in., as indicated
L' = torque span of beam, ft or in., as indicated
p = longitudinal steel ratio for bottom face of beam
p' = longitudinal steel ratio for top face of beam
p_t = transverse steel ratio
F_h = transverse force developed at yield in the transverse ties brought into action by a diagonal tension crack across the top of a beam, lb
F_v = vertical force developed at yield in the transverse ties brought into action by a diagonal tension crack on a side of the beam, lb

Each specimen is designated by a number and two letters followed by the number and size of the transverse ties within the torque span. The first number refers to the size of the longitudinal bars.

SPECIMENS AND TEST METHOD

Test specimens

All test specimens were 6 x 12 in. in cross section, 6 ft long, with a clear torque span of 5 ft. Three groups of six beams were cast, each group having a different amount of longitudinal steel. Within each group, the amount of transverse steel was varied from none (except for end reinforcement to prevent failure in the twisting device) to a maximum of #2 ties in pairs

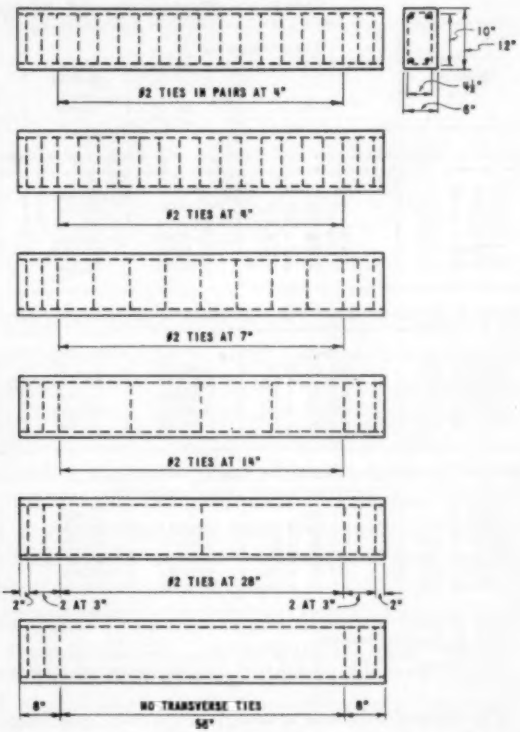


Fig. 1—Arrangement of steel for torsion tests. Longitudinal bars: #3 bars in Group 1; #4 bars in Group 2; #5 bars in Group 3. Transverse ties: Groups 1, 2, and 3 as shown

spaced at 4 in. as shown in Fig. 1. The arrangements of longitudinal and transverse steel, as well as all dimensions, are given in Fig. 1. Principal properties for all beams are listed in Table 1.

Materials, fabrication, and curing

All three groups, a total of 18 specimens, were cast from a single 2¼ cu yd delivery of ready-mixed concrete. The concrete had a nominal 28-day strength of 4000 psi, with a slump of 3 to 4 in., and was placed with a vibrator. The strengths of three 6- x 12-in. control cylinders, made at the beginning, middle, and end of the casting, were 3860 psi, 4000 psi, and 3910 psi, when tested at the mid-point of the testing period of the torsion specimens. Age at test was 24 days; cylinders were kept moist 1 week under plastic drop cloths and dried in air the remainder of the period, as were the beams.

Deformed bars meeting the requirements of ASTM A 15 and A 305 were used for all longitudinal reinforcement. The transverse ties were made from #2 plain reinforcing bars. Properties for all bars are given in Table 2 and Fig. 2.

Test set-up and procedure

All specimens were tested by means of torsion fittings designed for use with a universal testing machine (Fig. 3). The torque developed by the dead load of the fittings and specimen was calculated and included in all computations of torque applied to the beam.

Strain instrumentation consisted of SR-4 electric resistance gages placed through core holes at top, both sides, and bottom of the central transverse tie, and on each longitudinal bar adjacent to that tie. No attempt was made to obtain concrete strains in either tension or compression.

TABLE 1—PROPERTIES OF TORSION BEAMS*

Specimen No.	Transverse steel		Torque at first crack, in.-lb	Ultimate		Mode of failure†
	Ties	p_t		Torque, in.-lb	Detrusion, radians per in.	
Group 1—Longitudinal bars: top = bottom = 2-#3, $p = p' = 0.0031$						
3TR- 0	None	0	37,600	37,600	0.00010	DT
3TR- 1 #2	#2 @ 28 in.	0.00071	35,000	35,000	0.00010	DT
3TR- 3 #2	#2 @ 14 in.	0.00141	30,200	34,300	0.00042	DT
3TR- 7 #2	#2 @ 7 in.	0.00282	34,600	49,700	0.00166	DT
3TR-15 #2	#2 @ 4 in.	0.00494	35,000	61,700	0.00249	YP-DT
3TR-30 #2	2-#2 @ 4 in.	0.00985	35,000	76,000	0.00300	YP-DT-Shr.
Group 2—Longitudinal bars: top = bottom = 2-#4, $p = p' = 0.0055$						
4TR- 0	None	0	30,200	34,400	0.00005	DT
4TR- 1 #2	#2 @ 28 in.	0.00071	32,100	32,100	0.00010	DT
4TR- 3 #2	#2 @ 14 in.	0.00141	30,200	35,000	0.00082	DT
4TR- 7 #2	#2 @ 7 in.	0.00282	35,000	54,800	0.00123	YP-DT
4TR-15 #2	#2 @ 4 in.	0.00494	35,000	74,000	0.00195	YP-DT
4TR-30 #2	2-#2 @ 4 in.	0.00985	40,000	85,000	0.00200	YP-DT-Shr.
Group 3—Longitudinal bars: top = bottom = 2-#5, $p = p' = 0.0085$						
5TR- 0	None	0	33,800	33,800	0	DT
5TR- 1 #2	#2 @ 28 in.	0.00071	33,400	33,400	0.00015	DT
5TR- 3 #2	#2 @ 14 in.	0.00141	30,200	43,000	0.00150	DT
5TR- 7 #2	#2 @ 7 in.	0.00282	35,100	59,700	0.00117	YP-DT
5TR-15 #2	#2 @ 4 in.	0.00494	40,000	76,500	0.00202	YP-DT
5TR-30 #2	2-#2 @ 4 in.	0.00985	35,100	92,600	0.00170	YP-DT-Shr.

*For all beams: $b = 6$ in., $b' = 4.5$ in., $t = 12$ in., $t' = 10$ in., $L = 6$ ft, $L' = 5$ ft, $f_c' = 3920$ psi.
†DT = Diagonal tension
YP-DT = Yield point in steel and diagonal tension in concrete
YP-DT-Shr. = Yield point in steel, diagonal tension and shear in concrete

The angle of twist was measured over a 48-in. gage length by means of a troptometer having a minimum division reading of 0.02 radians, resulting in a detrusion of 0.000417 radians per in. per division of the scale. Since tenths of a division could be estimated, the minimum detrusion that could be detected was about 0.00004 radians per in.

The initial step in the test procedure consisted of applying a torque of about 5000 in.-lb in addition to the dead load of specimen and equipment, for the purpose of positioning and stabilizing the loading cables and torque drums. The initial strain and troptometer readings were recorded after this stabilizing torque had been applied. Torque was then applied up to about 15,000 in.-lb and released to 5000 in.-lb to check the operation of the troptometer and strain recording equipment. Following this, steel strain and troptometer readings were taken at specific load increments up to and well beyond the maximum torque. Crack inspections were made and crack patterns sketched at each load increment up to and including the maximum torque. The appearance of final fracture on each side of each beam was photographed after test.

TEST RESULTS

Principal test data

Principal test data are summarized in Tables 1, 2, and 3. The torque at first crack, ultimate torque and detrusion, and mode of failure are listed in Table 1. Properties of the reinforcing steel are provided in Table 2. Maximum steel strains, computed values of unit shearing stress and transverse steel stress at first crack and ultimate torque, values of the ratio of unit transverse shear to ultimate compressive strength, and certain plastic coefficients as computed from test data are in Table 3.

TABLE 2—PROPERTIES OF REINFORCING STEEL

Bar size	f_y, psi*	ϵ_y in/in	ϵ_{sh} in/in	Ultimate strength, psi*	Elongation in 8 in., percent	Specimen group, No.
#2	55,500	0.00185	0.0170	81,800	24	1, 2, & 3
#3	53,600	0.00179	0.0176	74,500	22	1
#4	41,000	0.00137	0.0192	71,000	27	2
#5	48,600	0.00162	0.0185	76,400	27	3

*Based on nominal area.

Behavior under pure torsion

The general behavior of a beam subjected to pure torsion is illustrated in Fig. 4 for four significant conditions. If the quantity of transverse steel is exceptionally low, or is spaced too widely, the beam will fail in diagonal tension at or somewhat above the torque at which the first crack develops, but without development of yield strains in either transverse or longitudinal steel. This condition is shown in Fig. 4(a) and is typical of the mode of failure of all nine of these beams having a value of p_t equal to or less than 0.00141 with a spacing greater than the depth of beam, and also of the beam in Group 1 having p_t of 0.00282 in combination with $p = p' = 0.0031$. These are the first three beams of each group listed in Tables 1 and 3.

When a beam contains equal volumes of longitudinal and transverse reinforcement, and the value of p_t is about 0.01 for these tests, the maximum torque is developed at the occurrence of a hybrid failure in diagonal tension and transverse shear preceded by yielding in both longitudinal and transverse

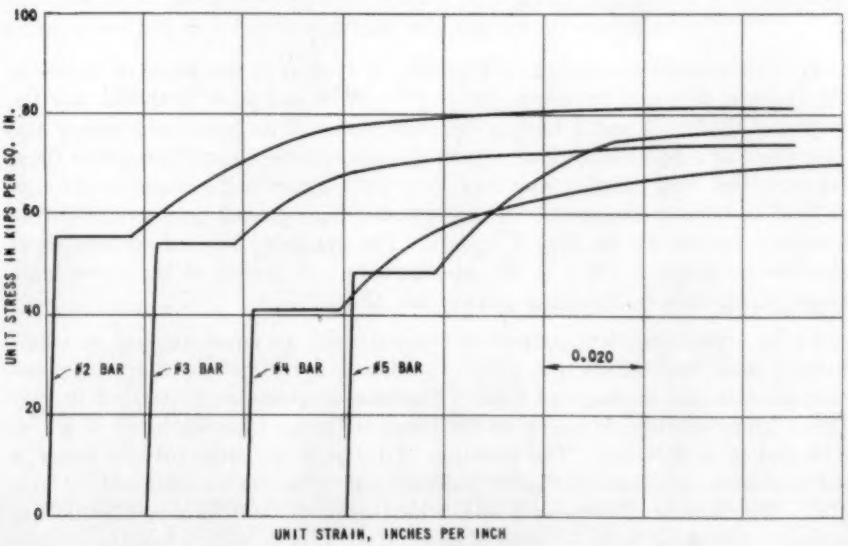


Fig. 2—Steel stress-strain curves

TABLE 3—COMPARISON OF STRAINS, STRESSES, AND COEFFICIENTS

Beam No.	$\frac{p_t}{p+p'}$	Maximum measured steel strain at ultimate, in. per in. $\times 10^6$		At first crack*		At ultimate torque				
		Long.	Trans.	v, psi	f_v, psi	v_{max},* psi	f_v,* psi	$k_v + k_h$‡	$k_v' = v_c'/f_v'$ ¶	$C \times 10^6$ **
Group 1										
3TR- 0	0	1930	No ties	354	—	354	—	—	—	7
3TR- 1 #2	0.11	1580	420	327	3850	327	3850	7.96	0.131	7
3TR- 3 #2	0.23	1270	1350	280	3300	318	3740	3.92	0.128	32
3TR- 7 #2	0.46	1550	1420	316	3720	455	5360	2.84	0.186	87
3TR-15 #2	0.80	4120	1880	313	3680	552	6500	2.00	0.232	104
3TR-30 #2	1.59	3670	1620†	300	3530	651	7670	1.23	0.285	102
			Avg	315	3710					
Group 2										
4TR- 0	0	1080	No ties	284	—	323	—	—	—	4
4TR- 1 #2	0.06	1290	260	300	3530	300	3530	7.30	0.129	8
4TR- 3 #2	0.13	1350	1020	280	3300	325	3830	4.00	0.131	61
4TR- 7 #2	0.26	1430	9020	320	3770	501	5900	3.13	0.205	58
4TR-15 #2	0.45	1910	2970	313	3680	661	7790	2.40	0.277	68
4TR-30 #2	0.90	4320	3120	342	4020	728	8560	1.38	0.318	61
			Avg	307	3610					
Group 3										
5TR- 0	0	880	No ties	318	—	318	—	—	—	0
5TR- 1 #2	0.04	680	580	312	3670	312	3670	7.60	0.125	12
5TR- 3 #2	0.08	760	1470	280	3300	398	4690	4.91	0.161	90
5TR- 7 #2	0.17	1270†	2490	321	3780	546	6440	3.41	0.224	51
5TR-15 #2	0.29	2410	2640	358	4210	684	8050	2.48	0.286	68
5TR-30 #2	0.58	1630	2240	300	3530	793	9330	1.50	0.347	48
			Avg	315	3700					

*Computed from Eq. (1) and (2).
†Yield strain was reached immediately following ultimate torque.
‡Computed from Eq. (8).
¶Computed from Eq. (10), and $k_v = 0.336$.
**Computed from Eq. (9).

steel. This condition, shown in Fig. 4(c), is typical of the mode of failure of the beam of Group 2 having $p_t/(p + p') = 0.90$ and $p_t = 0.00985$. For the beams of Groups 1 and 3 having the same value of p_t, maximum torque also developed at the occurrence of a hybrid diagonal tension and transverse shear fracture, but steel yielding was dependent upon the relative amounts of longitudinal and transverse steel. Torque-detrusion curves and photographs of the fractures are shown in Fig. 5 and 6. The tendency toward vertical shear fracture is clearly evident in the photographs. A sketch of the three main types of fracture is given for comparison (Fig. 7).

As the transverse steel content is lowered, with an equal amount of longitudinal steel maintained, the tendency toward a vertical shear fracture disappears and only a diagonal tension fracture develops, as illustrated in Fig. 4(b). This condition is typical of the beam of Group 1 having $p_t/(p + p') = 0.80$ and $p_t = 0.00494$. The lowering of $p_t/(p + p')$ substantially below a value of unity, produces the same general type of failure as exhibited by Fig. 4(b). The beams of Groups 2 and 3 having $p_t = 0.00282$ and 0.00494 are typical. For these tests, a value of about 0.0015 for p_t with a spacing greater than the depth of beam divided the conditions represented by Fig. 4(a) and

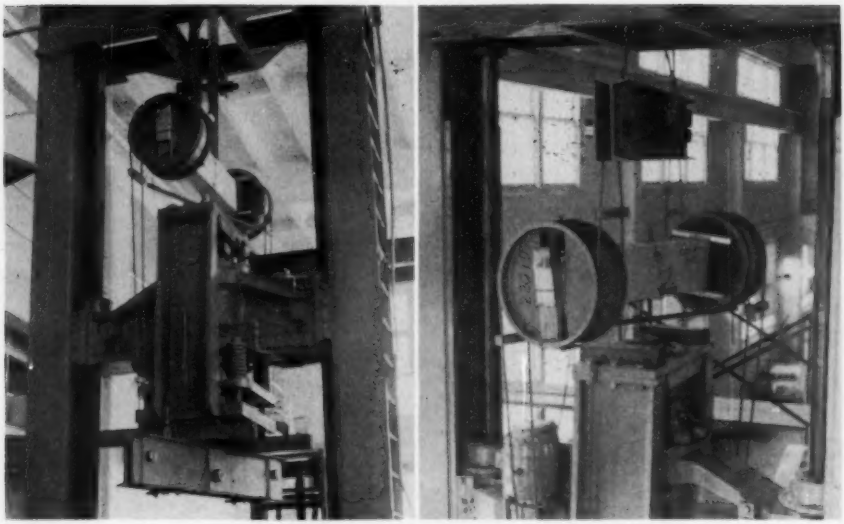


Fig. 3—Torsion equipment

4(b). Torque-detrusion curves and photographs of typical fractures in diagonal tension are provided in Fig. 5 and 6, respectively.

A possible upper limit, not developed in these tests, is illustrated in Fig. 4(d) for which it is assumed that the quantity of longitudinal and transverse steel is sufficient to develop a hybrid failure in vertical shear and compression without yielding of either transverse or longitudinal reinforcement.

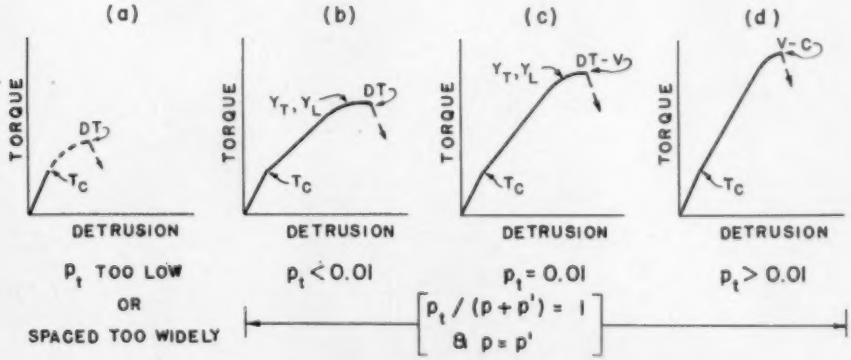


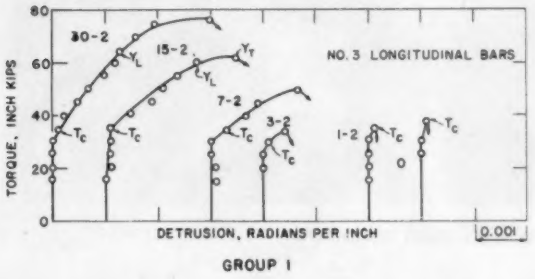
Fig. 4—Behavior under torsion

ANALYTICAL STUDIES

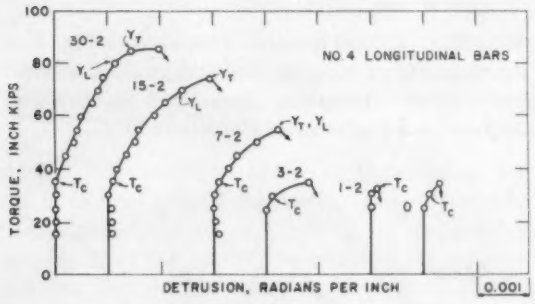
Theory

Expressions are already available for values of torque and detrusion when stress conditions are essentially elastic.[1,2,3] Only those equations directly useful to the analytical studies of these tests are given below, followed by a development of equations for plastic ultimate torque and detrusion.

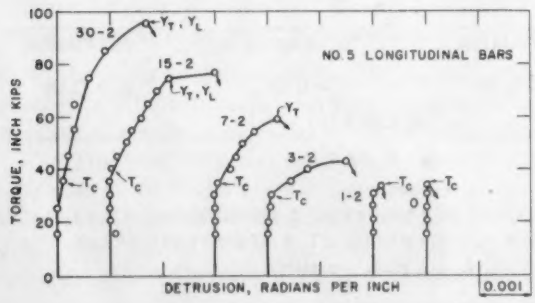
Elastic equations—The development of an expression for the resisting torque of rectangular reinforced concrete sections in the elastic range has been thoroughly presented by H. J. Cowan, of the University of Sidney, with



GROUP 1



GROUP 2



GROUP 3

Fig. 5—Torque-detrusion curves

accompanying equations for steel stress and detrusion.[1,2,3] The following equations have been selected from his work, and adjusted for the vertical and horizontal reinforcement used in these tests.

$$T_s = \alpha\, b^2\, t\, v_{max} + \frac{\lambda}{2}\, b'\, t'\, \frac{A_v}{s}\, f_v \quad \ldots\ldots (1)$$

$$f_v = 2n\, v_{max}/\sqrt{2} \quad \ldots\ldots (2)$$

$$\phi_s = \frac{T_s}{\beta\, G\, b^3\, t} \quad \ldots\ldots (3)$$

For the cross-sectional dimensions of the test beams: $\alpha = 0.246$, $\beta = 0.229$, and $\lambda = 1.614$.[1,2,3]

Also, for equal volumes of vertical and horizontal reinforcement (i.e., balanced steel content), in which A_L = total area of longitudinal steel placed symmetrically within the periphery of the reinforcement cage:

$$\frac{A_L}{A_v} = \frac{2\,(b' + t')}{s} \quad \ldots\ldots (4)$$

The above equations may be applied to ultimate torque conditions as long as brittle fracture dominates failure. However, if the proportions and quantities of transverse and longitudinal steel are such that the plastic properties of the steel in the yield strain range dominate failure, the tension contribution of the concrete recedes in significance and plastic equations should be used to define failure.

Plastic equations—If the quantity of longitudinal steel is equal to or greater than that given by Eq. (4), the transverse ties will provide the transverse components required to resist the diagonal stresses at cracks formed on the sides and upper and lower surfaces of the beam. The vertical force developed at yield in the transverse ties brought into action by a diagonal tension crack on a side of the beam is

$$F_v = \frac{A_v\, f_y\, t'}{s} \quad \ldots\ldots (5)$$

Also, the transverse force developed at yield in the transverse ties brought into action by a diagonal tension crack across the top (or bottom) of the beam is

$$F_h = \frac{A_v\, f_y\, b'}{s} \quad \ldots\ldots (6)$$

These forces act with the transverse component of the diagonal compression in the concrete on the opposite face to form the internal resisting couple at

plastic ultimate torque, resulting in the following expression for torque

$$T_p = F_v k_v b' + F_h k_h t' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which $k_v b'$ and $k_h t'$ represent the lever arms in terms of the dimensions of the reinforcement cage. Substituting Eq. (5) and (6) in Eq. (7),

$$T_p = (k_v + k_h) \frac{A_v f_y}{s} b' t' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

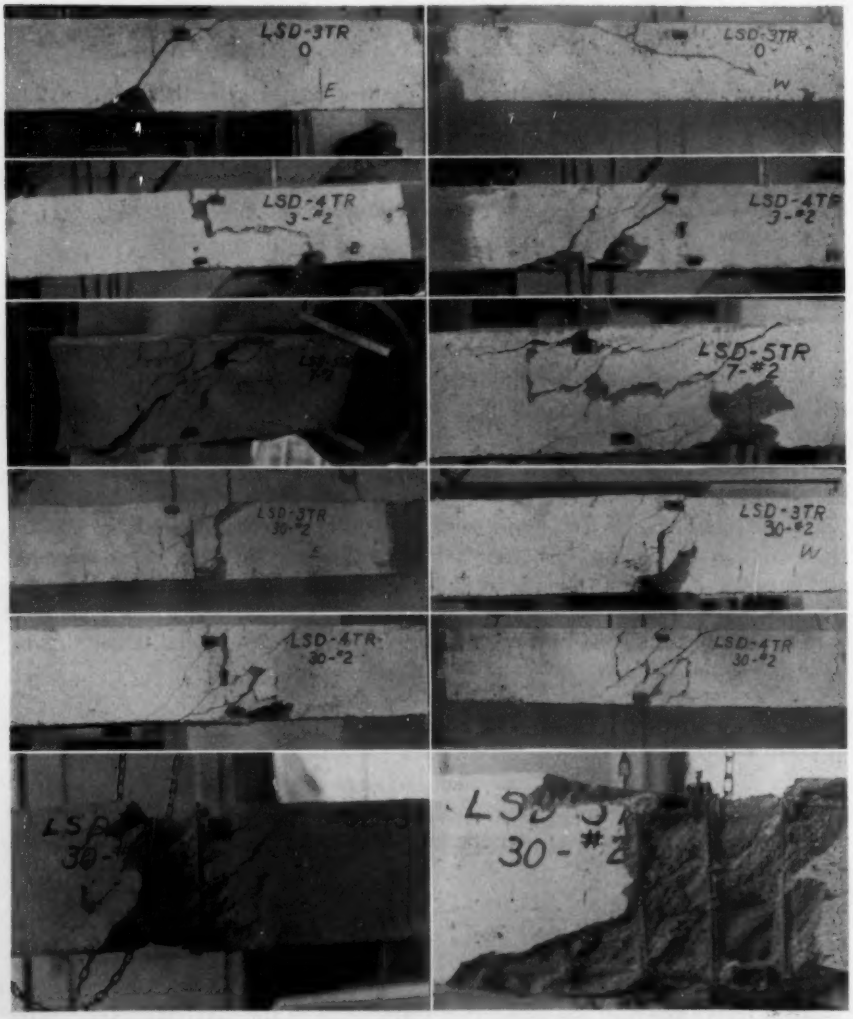


Fig. 6—Typical fractures photographed from opposite sides of the beam

Comparison of Eq. (8) with Eq. (1) shows that the concrete is not relied upon for tensile resistance for plastic conditions, and that the form of the expression for the torque supplied by the steel is identical. The coefficient $(k_v + k_h)$ is tabulated (Table 3) for the data of these tests.

Lack of any marked region of plastic rotation during which the torque was constant limits the maximum angle of twist to that developed at maximum torque. The accumulation of plastic detrusion is very similar in nature to the accumulation of flexural unit rotation in the vicinity of a plastic hinge, in that it is a physical characteristic of the member, and must be determined experimentally. The spread of yield on either side of a crack, the physical qualities of the concrete and steel, and the quantity and arrangement of the longitudinal and transverse steel all enter into the accumulation of twist angle at maximum torque. Test data were not sufficient to justify firm conclusions with regard to a quantitative relationship between twist angle and the other variables. If the form of Eq. (3) is assumed to hold at plastic ultimate torque, then

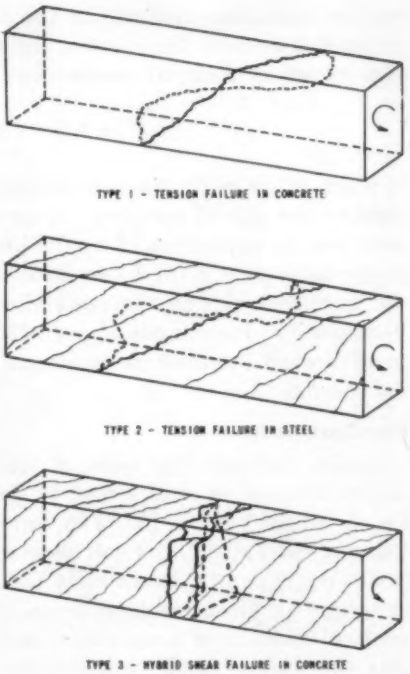


Fig. 7—Types of torsional fractures

$$\phi_p = C \frac{T_c}{b^3 l} \quad \ldots\ldots\ldots\

may be used that conforms to the first term of Eq. (1), i.e., sufficient steel is supplied to force the concrete throughout the entire area of the reinforcing cage to act in diagonal compression and transverse shear, then

$$T' = k_c\,(b')^2\,t'\,v_c' = k_c\,k_c'\,(b')^2\,t'\,f_c' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which $k_c' = v_c/f_c'$, the ratio of transverse shearing stress to ultimate compressive strength of concrete. k_c for a rectangular section in the semiplastic state may be approximated by the average of the elastic value α and the fully plastic value $(3 - b'/t')/6$ (see Reference 3 or 12). For the dimensions of the test specimens, this approximation for k_c would be $(0.246 + 0.425)/2 = 0.336$. Values of k_c', based upon $k_c = 0.336$ are provided in Table 3, showing the trend toward complete development of the transverse shearing strength of the concrete.

Significant factors

Initial cracking—The start of tensile cracking, as indicated by the first abrupt increase in steel strain, is shown on each torque-detrusion curve of Fig. 5 by the symbol T_c. The corresponding values of torque are listed in Column 4 of Table 1, and unit shearing and transverse steel stresses computed from Eq. (1) and (2) are listed in Columns 5 and 6 of Table 3. The variation in torque at which cracking began exhibits no relationship to the actual or relative amounts of longitudinal and transverse reinforcement. It is clear that the start of tensile cracking corresponds to the failure of unreinforced concrete in torsion, with the average unit shearing stress at 312 psi.

Torque and reinforcement—The transverse ties were spaced at 28 in. and 14 in. in order to provide values slightly less than $2b' + 2t'$ and $b' + t'$, respec-

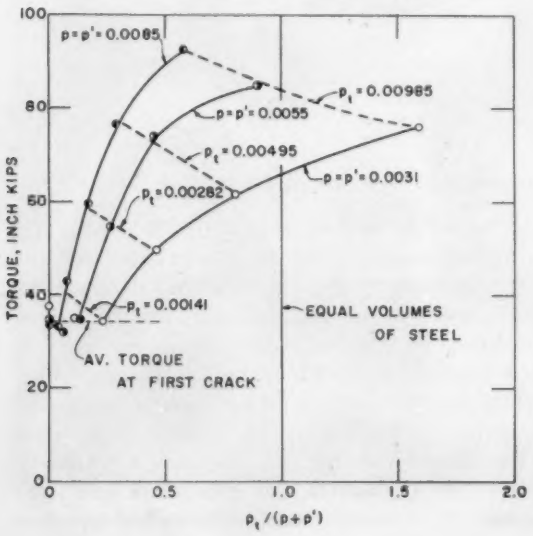


Fig. 8—Torque versus steel ratios

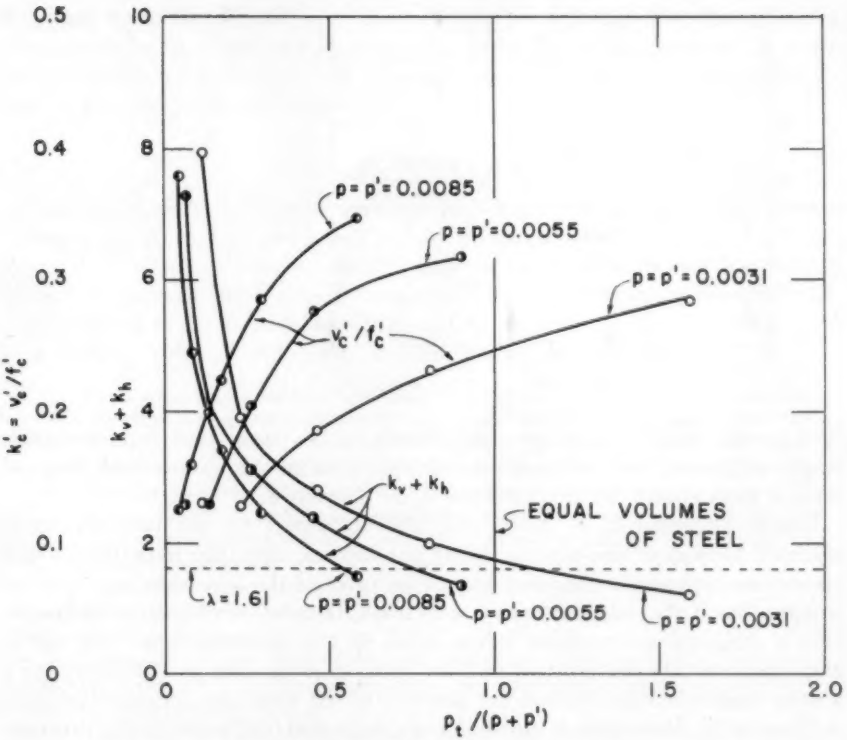


Fig. 9—Plastic coefficients versus steel ratios

tively. Referring to Fig. 8, it may be seen that even the smaller spacing ($p_t = 0.00141$) did not materially increase the torque above that for an unreinforced section, excepting for a slight improvement for $p = 0.0085$. Also, for a constant value of p_t greater than 0.00141, the torque capacity was improved by increasing p and p' by equal amounts, even though the ratio of transverse to longitudinal steel was below that for balanced volumes. It is evident from Fig. 8 that increasing the ratio of transverse to longitudinal steel by decreasing only the longitudinal steel will decrease the torque capacity, but increasing the ratio by varying only the transverse steel will improve torque capacity.

Calculated v_{max} and f_v—The maximum unit shearing stress and corresponding steel stress as computed from Eq. (1) and (2) are listed for each beam in Table 3. The values of v_{max} for the first three beams of each group, failing by diagonal tension without yielding of the steel, are practically equal to the diagonal tension capacity of the concrete. However, it is evident from a comparison of the maximum measured strains of Column 4 with the values of f_v in Column 8 that the transverse steel takes a greater portion of the diagonal tension than indicated by elastic theory, as the concrete becomes cracked to a deeper extent within the beam. For all but one of the last three beams of

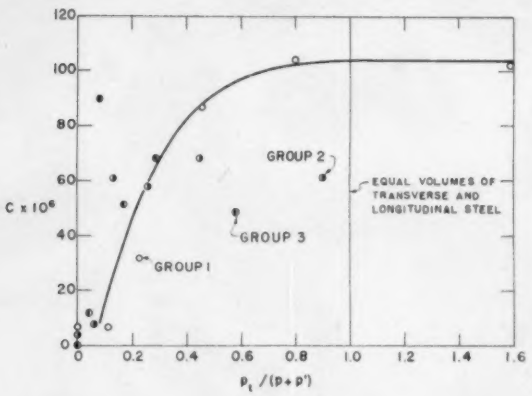


Fig. 10—Detrusion coefficient

each group, yield strains were developed in the transverse reinforcement, clearly indicating that the steel was carrying a far greater share of the diagonal tension than shown by the values of f_v in Column 8.

Plastic coefficient $k_v + k_h$—If the state of stress at ultimate torque is assumed typical of the plastic theory for flexure, then the longitudinal and transverse steel will act to carry the entire value of the horizontal and vertical components of the diagonal tension at a crack, thereby forming a torque couple with a diagonal compression stress block on the opposite face. Eq. (8) is developed on this assumption, and the term $k_v + k_h$ may be considered as a plastic coefficient that defines the position of the resultant compressive force relative to the dimension of the reinforcing cage, and for the particular arrangement of steel used in these tests. Values of $k_v + k_h$ computed from Eq. (8) with the test data are listed in Table 3 and plotted against $p_t/(p + p')$ in Fig. 9. A transition from the elastic state [as represented by Eq. (1)] to a condition approximating the plastic state seems clear.

v_c'/f_c'—The transition suggested above results in an effectiveness of only the concrete within the reinforcing cage, and the upper limit of torque may be reached by failure of the concrete in diagonal compression or transverse shear or a hybrid of the two. Eq. (10) expresses this possibility. Values of k_c', the ratio of ultimate transverse shearing stress to ultimate compressive strength, were computed from Eq. (10) with the test data and a value of k_c equal to the average of the fully elastic and fully plastic states, i.e., $\alpha/2 + (3 - b/t)/12$. Table 3 and Fig. 9 provide these values of k_c'. As the ratio $p_t/(p + p')$ is increased, the greater the liklihood of full development of the transverse shearing strength of the concrete, providing that the total steel content is sufficiently high. The type of final failure at the ultimate torque for the last beam of each group ($p_t = 0.00985$) is shown in Fig. 6, in which the vertical fracture may be noted. For all other beams, diagonal tension fracture dominated.

Detrusion coefficient—As stated just preceding Eq. (9), test data on angle of twist were hardly sufficient to warrant conclusions, nor were they consistent

enough to provide definiteness in the trend of the coefficient C of the equation. On the other hand, a plot of the values of C (from Table 3) against $p_t/(p + p')$ provides evidence that C may approach a constant value for beams approximating the plastic state of stress at failure (see Fig. 10).

SUMMARY

Eighteen rectangular reinforced concrete beams were tested in pure torsion to determine interaction between the longitudinal and transverse steel and the concrete at ultimate torque. Strain gage locations were selected to obtain the earliest possible indication of initial cracking and yield strain development.

The results of these tests, in which the longitudinal steel ratios were $p = p' = 0.0031$, 0.0055, and 0.0085, may be summarized as follows:

1. For a spacing of transverse ties equal to or greater than $b' + t'$ ($p_t \leqq 0.00141$), failure occurred in diagonal tension at or only somewhat above the torque at which the first crack developed, without yield strains in either transverse or longitudinal steel.
2. For all beams, the start of tensile cracking corresponded to the failure of unreinforced concrete in torsion, at an average unit shearing stress of 312 psi.
3. For the beams containing the highest ratio of transverse steel, $p_t = 0.00985$, the maximum torque was developed at the occurrence of a hybrid failure in diagonal tension and transverse shear, preceded by yield strains in the steel.
4. Beams with transverse steel ratios greater than 0.00141 but less than 0.00985 exhibited diagonal tension failures at ultimate torque, preceded by yield strains in the steel in all but one case.
5. Increasing the total longitudinal steel content, for a fixed value of p_t, improved the torque capacity even though the ratio of the transverse to longitudinal steel was below that for balanced volumes.
6. Increasing the ratio of transverse to longitudinal steel by increasing only the transverse steel results in improved torque capacity. On the other hand, increasing $p_t/(p + p')$ by decreasing only $p + p'$ results in a decrease in torque capacity.
7. These tests show substantial evidence of a transition from elastic to plastic states of stress at ultimate torque as the value of $p_t/(p + p')$ approaches unity, for $p = p' = 0.0031$ or greater. There is also evidence that the upper torque capacity is limited by the transverse shear strength of the concrete.
8. Detrusion data were scattered, but to some extent support the possibility of a constant plastic detrusion coefficient for beams having properly proportioned transverse and longitudinal reinforcement.

ACKNOWLEDGMENTS

This project was made possible by a grant from the National Science Foundation for research on reinforced concrete connections in the plastic range. A part of the problem of connections concerns the transfer of torsional moments from beams to columns, for which these tests provide basic knowledge.

The tests were carried on in the testing laboratory of the department of engineering mechanics of the University of Nebraska, through the courtesy of Prof. E. J. Marmo, chairman of the department.

Acknowledgment is given to Kuno Smits, civil engineering student, for his assistance in the making and testing of the specimens, and in computing and drafting.

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Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1958, for publication in the June 1958 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

Inspection and Control of Haydite Concrete 357
Problems and Practices 359

Inspection and Control of Haydite Concrete

By E. L. HOWARD and M. B. JONES*

The data in this report are from tests of Haydite, sampled and tested from one source delivered to two batch plants. Transit-mixed concrete was hauled from the two plants to three large and several small jobs. Commercial labs designed and inspected the concrete used on the larger works. The inspectors, faced with the variables common to lightweight concrete limited their control to the adding of mixing water. Their data are not available for this report.

All of the problems normal to regular concrete plus certain special problems plague the inspector of Haydite concrete. Uniformity of slump and accuracy of yield cause the most concern. Mortar-aggregate separation when the concrete is too wet causes the finishers much trouble. Truck standing time while water is added to dry loads slows placing and adds cost. A short yield adds to costs when the yardage is made up, while a long yield means reduced cement content per cu yd. A chief source of trouble is the high absorption of lightweight aggregates. If the material is used as delivered to the batching plant the moisture may vary from 0 to 10 percent. This variance makes the batching of mixing water mere guess work.

Adding confusion to the inspector's test data is the inadequacy of his tools for a job in lightweight concrete. The slump cone is not as accurate as the Kelly meter in measuring consistency, and neither is quite satisfactory for the job. The slumps are usually measured by "eyeball" by those who too often are unfamiliar with the problem of the "floating aggregates." Sieve analysis of the lightweight aggregates has small meaning when shown in the standard percentages by weight, as each of the particle sizes has a different specific gravity. Day to day variations in the specific gravities of the delivered materials quite often out-date a mix proportioned from samples taken before the job begins.

Limited manpower for job inspection made it imperative for this office to plan a control system operative from a central lab and effective for all jobs receiving concrete from the two batching plants. The specific gravities, absorptions, and moisture tests reported here were taken to furnish data for quality control of lightweight concrete with the very minimum of inspection.

To take the guesswork out of the adding of mixing water the aggregates were saturated on arrival and stored in the batch plant bins with about the same free moisture as occurs in natural concrete aggregates. When the absorption by the aggregates of mixing water was not a problem, the primary variable affecting consistency and yield was eliminated. Only when the aggregate surface moisture was less than 0 percent did water control problems arise.

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*Chief Testing Engineer and Assistant Testing Engineer, respectively, Pacific Cement & Aggregates, Inc., San Francisco, Calif.

TABLE 1—PROPORTIONS FOR HAYDITE CONCRETE MIXES

Material	Amount per cu yd of concrete, lb, saturated, surface dry	Volume at minimum specific gravity, cu ft	Volume at maximum specific gravity, cu ft	Volume at average specific gravity, cu ft
Pacific Cement & Aggregates Mix No. 1693				
¾-in. Haydite	540	6.18	4.97	5.51
⅜-in. Haydite	440	4.38	3.73	4.03
Fines, Haydite	440	3.79	3.30	3.52
Commercial sand	840	5.00	5.00	5.00
Cement	564	2.87	2.87	2.87
Water	312	5.00	5.00	5.00
Darex	5 oz	1.08	1.08	1.08
Total, cu ft		28.30	25.95	27.02
acific Cement & Aggregates Mix No. 1694				
¾-in. Haydite	540	6.18	4.97	5.51
⅜-in. Haydite	440	4.38	3.73	4.03
Commercial sand	1440	8.55	8.55	8.55
Cement	564	2.87	2.87	2.87
Water	312	5.00	5.00	5.00
Darex	5 oz	1.08	1.08	1.08
Total, cu ft		28.06	26.20	27.04

Variations in concrete yield follow the changing specific gravities. Table 1 illustrates the seriousness of this variable. On jobs of long duration, the standard mix designs shown average out very well as to yield. A small job going during a period of high or low specific gravity might be 5 percent long or short. In either case the settlement of yardage delivered is a problem for all concerned. The mixes proportioned by commercial labs for the jobs represented in this report averaged 27.7 cu ft to the designed yard. These improperly proportioned mixes were set up on samples that were not representative of the average material used during the work. Variations in specific gravities may occur suddenly and often, necessitating adjustments of batch weights.

The unit weight tests of fresh concrete are made too late for proper control. Samples representative of batch plant bin material can be tested for specific gravity, moisture, etc., prior to concrete placing, in less than 30 min. For these data the pycnometer

TABLE 2—SPECIFIC GRAVITY, ABSORPTION, AND MOISTURE DATA OF SEVERAL SIZES

Property	¾-in. Haydite	⅜-in. Haydite	Fine Haydite
Specific gravity			
number of samples	56	58	27
average	1.57	1.75	2.00
deviation	0.17	0.14	0.14
Absorption			
number of samples	29	27	14
average	12.4 percent	12.2 percent	13.4 percent
deviation	3.1	4.8	2.4
Moisture			
number of samples	43	53	
average	1.7 percent	2.9 percent	
deviation	0.41	1.3	

provides a fast accurate method of test. Coarse aggregate can be weighed wet, quickly surface dried, weighed, and put in the pycnometer. Five minutes lab work for each aggregate yields the information needed for batch weight adjustment. Specific gravity of fine aggregates requires more time but a moisture meter in the weigh hopper gives the operator instant check on moisture changes. (It is wise to use as little of the lightweight fines as the specified concrete unit weight will allow.) With these tests, uniformity of yield becomes a possibility.

As long as deviations of 0.15 and more (Table 2) occur in the specific gravities of Haydite materials, the ready-mix concrete dealer will need a qualified inspection of the aggregates to keep the lightweight mixes in proper control and adjustment. The minimum inspection would be tests for moisture and specific gravity of each shipment. Perhaps the day will come when lightweight aggregate producers will establish quality control measures as rigid as those set for sand, gravel, and cement companies.

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

To some, the answers will seem simple and obvious; others may prove to be extremely controversial.

All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL *readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.*

Q. The accompanying drawings show typical cracks which occurred only in corner columns of a three-story project we have been asked to investigate. The structure has outside columns about 14 x 20 in., inside columns about 18 in. square; outside spandrel beams about 20 x 24 in., inside one-way beams about 16 x 22 in., 8 in. one-way slabs. Building is four bays by five bays, corner bays about 22 ft square.

All concrete work and reinforcing steel conform with ACI 318-51 and the ACI detailing manual (ACI 315-51). Reinforcing bars conform with ASTM A 305-49. Foundations are spread footing type resting on a hard limestone bed with a bearing capacity of 20,000 psf. Floors and roof are designed for 175 psf live load.

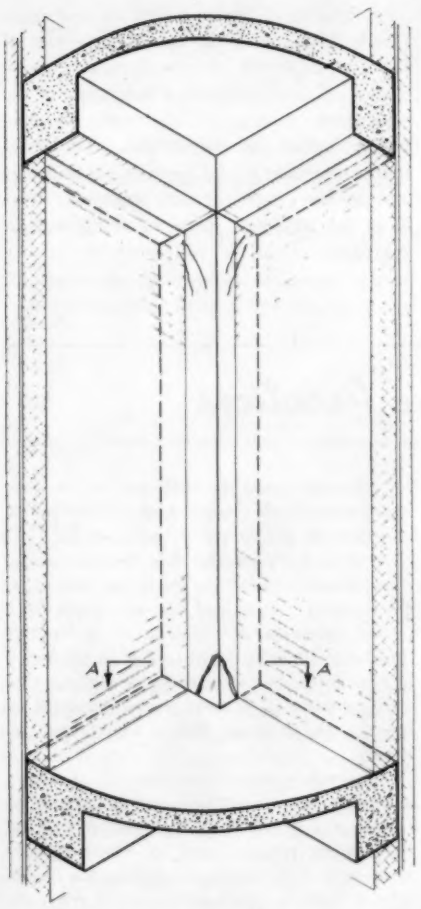
Allowable stress used for all column concrete was 3750 psi at 28 days; all other concrete had an allowable stress of 3000 psi at 28 days. Reinforcing steel stresses were 20,000 psi in tension and 16,000 psi in compression.

At present, the existing three floors are loaded approximately ⅓ to ½ of the design capacity of the structure. Provision has been made for a lateral extension of building in a westerly direction, this being taken care of in the design of outside column row at the extension. The west curtain wall is wood frame supported on concrete and veneered with 4 in. of brick on outside.

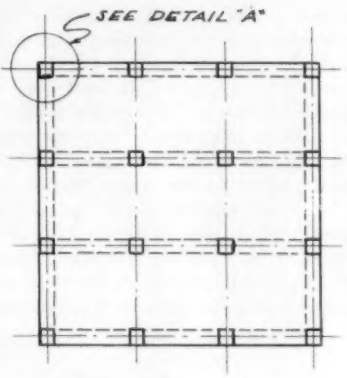
The cracks are most prominent in column at southeast corner of building, particularly on the second and third floors. Those at base of second-story column meet as shown in the drawing and the triangular section has broken off. Cracks in third-story column are more nearly vertical, and one stops at receptacle outlet. The cracks at tops of columns are finer and the surfaces above cracks have quite commonly been pushed out beyond face below. There are no cracks discernible in floor beams and slabs.

We understand that the concrete was placed in cold weather and that floor forms were stripped from 6 to 9 days after concreting. The concrete mix was the same as for similar buildings in which such cracking has not occurred.

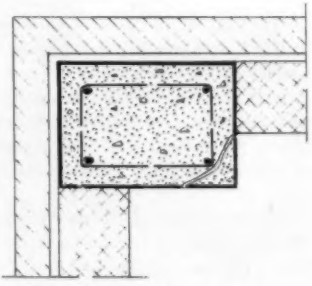
A part of the loose triangular section of concrete from southeast third-story column is very hard and, much of the limestone aggregate at break shows clean fractures. We would appreciate any information as to the cause of these cracks which evidently developed before completion of the building.



DETAIL "A"



FLOOR PLAN



SECTION A-A

A. There are at least two possible hypotheses to explain this failure:

1. The column has cracked due to a corner moment developed as the result of two-way slab action (whether designed as a two-way slab or not) which produced a negative moment parallel to the diagonal. This hypothesis seems to fit the crack pattern particularly well, and would also explain the relative movement of the faces of the column above and below the cracks. This explanation would seem particularly possible, if the column is lightly reinforced and if the shoring was removed from the floor before all column dead load was in position.

2. Since the distress did not appear in similar buildings of presumably identical structural design, one should take into account differences in (a) construction methods, (b) conditions of weather, temperature, etc., and (c) possible differences in materials, slump, shrinkage, and thermal coefficients of the concrete. This latter possibility might lead to the first hypothesis if construction methods varied, or to explanation as a negative volume change in the floor slab after the roof slab was

cast. Note that the longest dimension in which shrinkage could occur is on the diagonal, approximately at right angles to the crack (see plan) and would probably be of *much* higher order in the corner columns than in any of the intermediate columns.

* * *

Q. In the same three-story building in which concrete corner columns cracked (see preceding question), a large percentage of the 8 in. thick concrete masonry back-up in exterior walls appears to have moved out from a slight amount to a maximum of nearly ⅜ in., ¼-in. movement being quite common.

This apparent wall movement has occurred in all corner bays and in general is greater there. There has been no such movement from the floor in the first story supported on the ground.

The apparent back-up movement has left a corresponding space between the 2-in. floor topping; which was placed after the masonry walls were built, and the wall. In at least one case the bond of asphalt tile base to wall has taken tile out with wall, leaving a space between first and second rows of tile. The floor topping and finish have remained in contact with partitions.

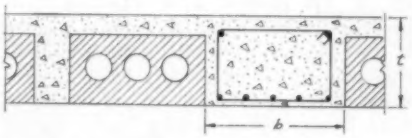
A. A later inspection of the exterior column opposite the principal inner corner crack, through a temporary opening in the exterior brick veneer has shown cracks in the exterior of the concrete column at this point which indicate that column cracks and apparent wall movement are both due to abnormal shrinkage in the concrete floor construction. This shrinkage would naturally be greatest at the corners because of the greater distance on the diagonal from center of floor construction which consists of 8-in. solid slabs about 90 x 100 ft with 20 x 17-in. concrete beams underneath supported on five rows of six columns each.

The shrinkage in concrete has evidently drawn the exterior spandrel beams inward, and a wedging of the masonry wall between columns has apparently held the back-up in place, allowing the floor construction and finish to move away from them, although it appeared that the back-up had moved out. This slippage of back-up on floor construction was facilitated by the coated metal spandrel flashing under the back-up.

The lack of any apparent movement in exterior brickwork from ground and roof bears out this theory which is the most logical explanation of these conditions advanced to date.

* * *

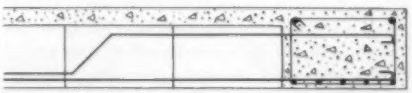
Q. What is the maximum width which may be considered effective in a shallow beam supporting loads concentrated on a line, such as partitions? The problem arises most frequently in practice in the design of uniform depth, one-way, joist floors in which a solid section formed by omitting fillers is designed as a beam.



t = 8-12 in., usually; $b = 2t$ (preferred); $b = 3t$ maximum

A. Usual practice in this area (Peru) is to limit the width of these "flat beams" to a maximum of three times the thickness. A limit of two thicknesses is preferred. The best practice is to provide at least nominal compression reinforcement (two #5 top bars) and nominal web reinforcement (#3 closed stirrups, 12-in. spacing) throughout the length of the flat beams. Bond stress is particularly critical and should always be checked.

In another case—where the shallow beam is perpendicular to the floor reinforcement—the beam may support the slab as well as



partitions above. In our experience, the effective width of beam is the same as in the previous example. A criticism of this practice on a theoretical basis would be appreciated.

JAIME DE LAS CASAS
Lima, Peru

* * *

Q. What type of form oil will give best results with steel forms? Is the choice of form oil related to the occurrence of air bubble holes on the concrete surface, and how can they be eliminated?

A. Apparently straight petroleum oils work best only on wood forms. It seems that specially compounded petroleum oils are

required for steel forms. Vegetable oil, synthetic castor oil, silicone, and graphite treated oils are used in various successful commercial compounded form oils for steel.

Air bubble holes as surface defects are encountered with any smooth, impermeable form surface like steel, plastic, or concrete. Absorbent paper and muslin liners are used for some work where the importance of eliminating these surface defects justifies the cost. ACI Committee 614 recommends up to 100 percent more vibration than necessary to consolidate the concrete, placing in more shallow layers, and hand spading to eliminate air bubble holes. Neither extra vibration nor spading will remove air-bubble holes from surfaces molded under sloping forms, only vacuum forms or absorptive form lining will remove air-bubble holes from such surfaces.

* * *

Q. A horizontal crack opened (widest at midbay) in the joint above the bottom course of concrete block partitions. The condition occurred as a typical pattern in a multistory reinforced concrete frame building. The partitions generally follow column center lines. The floor is a 10 in. thick flat plate designed for 100 psf live load and the spans are 28 ft. What caused the cracking and what remedy can be applied? What precautions would help to avoid this condition on future projects?

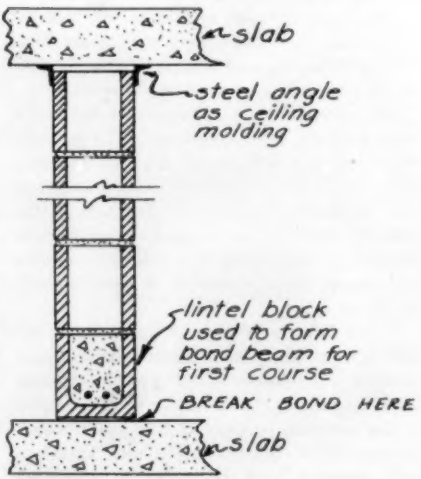
A. The cause of the cracking is the relative flexibility of the slab compared to the wall. A relatively rigid masonry partition wedged into a deforming structure is not going to conform itself to movements in the structural frame without cracking. The slab can deflect without visible distress, but the stiff wall is unable to follow it. Until it cracks, the wall acts as a deep beam spanning between columns. After cracking, a plain masonry arch develops within the wall panel bridging over the center portion.

The simplest remedy after the fact is to fill the crack with mastic caulking compound. As the deflection of the slab increases with creep the mastic will stretch, and so perform better than mortar pointing as a remedy.

To avoid this condition on future work three possible procedures are;

1. Where its appearance is not objectionable the simpler precaution is to build an open, vertical "control joint" at the midpoint of the wall between columns. Caulking this joint with mastic should be delayed as long as possible to allow creep deflection before caulking.

2. For remodeling involving new partitions on existing thin or unduly flexible flat plate floors or where a vertical joint or a beam below slab is undesirable, a heavily reinforced bond beam bottom course can provide necessary tensile resistance to allow the wall to act as a beam. Bond of the bottom mortar joint to the floor should be broken by paper. A horizontal crack may open at this bottom joint but it can easily be concealed by the base molding. If the floor is unduly flexible, center shoring until reinforced masonry partition is completed would be an advisable precaution.



Heavily reinforced bond beam bottom course. Steel angle used as ceiling molding to conceal top joint and provide lateral bracing at top

3. A stiffening beam in the floor slab below the line of each masonry partition may be embodied in the design. This was done in earlier flat slab construction, along with the use of drop panels and capitals, making the total structure relatively stiffer.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Contribution to the design of skew slab bridges (in German)

H. Voot, *Bauplanung-Bautechnik* (Berlin), V. 11, No. 2, Feb. 1957, pp. 83-86

Reviewed by I. M. Viest

Deals with skew concrete slab bridges reinforced in a direction other than that of principal moments. The ratios of the moments in the directions of the reinforcement to the principal moments are given in tabular form for various degrees of skew and for several different ratios of the principal moments. Use of the method is illustrated with a numerical example.

Reconstruction of bridges in France during and after the 1939-45 war with particular reference to site organization and conditions

Rene Breffeil, *The Structural Engineer* (London), V. 34, No. 10, Oct. 1956, pp. 338-347

Reviewed by C. P. Siess

Reconstruction of three railway bridges described. The Longeray Viaduct has three near-Gothic arches of 198, 230, and 178-ft spans with the deck supported only at piers and arch crown. The Montlouis Bridge is a multispan arch structure rebuilt in concrete on the original piers. The Nogent-sur-Marne Viaduct consists of three parabolic concrete arches of 260, 230, and 220-ft spans, supporting continuous box girders.

Erembodegem Bridge (in French)

J. Capel, *Bulletin*, CERES, Liège, V. 7, 1955, pp. 441-452

Reviewed by E. D. Rolin

A continuously prestressed bridge of two 87-ft exterior spans and one 174-ft interior span. The girders are 5 ft 11 in. deep. The interior supports consist of a pair of prestressed inclined columns under each girder, in the shape of an upright V with an included angle of 30 deg and articulated at top and bottom. Avowed advantages of the system are: (1) the V reduces slightly the clear span; (2) the reactions being inclined, a horizontal load is induced in the larger center span; (3) the great rigidity of the portion of the beam across the mouth of the V creates high negative moments.

Construction

Elliott Secondary School, Putney

Walter C. Andrews, *The Structural Engineer* (London), V. 34, No. 11, Nov. 1956, pp. 404-414

Reviewed by C. P. Siess

Describes many interesting and some novel features of multistory classroom building, gymnasia, and assembly hall, including: various framing plans studied; concrete columns cast in precast concrete shells; cantilevered stairs; undulating shell roof; prestressed girders of 73-ft span, cast in three pieces, prestressed, and lifted into place; load test on stairway; and proof test of a prestressed beam having cracks at junction of bottom flange and web.

New wide span roof

Prefabrication (London), V. 4, No. 45, July 1957, pp. 395-397

A factory building in England employed a novel wide-span roof. The roof is basically a north light structure consisting of concrete shells spanning between steel arch trusses. The concrete shell, approximately 3 in. thick, is post-tensioned through a high-tensile tie-rod attached at the springing of the steel arches. The concrete shell spans 20 to 25 ft from the top chord of one arch to the bot-

A part of copyrighted Journal of the American Concrete Institute, V. 29, No. 4, Oct. 1957, *Proceedings* V. 54. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members that cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the 'Current Reviews" section are available from ACI headquarters on request.

tom chord of the next. Vertical glazing is used to fill the triangular panels formed by the truss members. The steel arches are prefabricated in standard unit sizes, 80, 100, 120, 150 ft, to a maximum of 210-ft span. The complete roof offers considerable advantages in its capacity for industrial loads suspended from below, in its lightweight acoustic qualities, light reflection qualities, and reduced volume for heating purposes.

Tunnel linings

O. Dawson, *Civil Engineering and Public Works Review* (London), V. 52: No. 609, Mar. 1957, pp. 317-319; No. 610, Apr. 1957, p. 430-433

A review of methods in use for lining large tunnels. Due to shortages and high prices of cast iron forms, alternative precast concrete and cast-in-place linings have been developed. Four systems of precast concrete block linings are described. Segmental and prestressed concrete linings are also pictured and described.

Reconstruction of the auditorium floor at the Royal Opera House, Covent Garden

C. H. Hockley, *The Structural Engineer* (London), V. 34, No. 12, Dec. 1956, pp. 435-442
Reviewed by C. P. Siess

Detailed description of installation of new floor consisting of hardwood strip flooring on precast, prestressed concrete joists supported on steel beams and columns.

Design

Reinforced concrete designer's handbook

Charles E. Reynolds, Concrete Publications, Ltd., London, 5th Edition, 1957, 343 pp., $4

A new edition incorporating recommendations of the British standard code of practice for reinforced concrete in buildings issued June, 1957. The ultimate load method of design which was permitted by the latest British standard code is included in the handbook for the first time. The material is presented in four parts: information and explanatory notes in Part 1; tables for use in design in Part 2; examples of particular designs requiring reference to more than one table in Part 3; specifications and quantities in Part 4, including design. Design includes loads and pressures, calculation of bending moments and forces for particular structures, and general calculation of stresses and design of members. In addition to the section on specifications, an appendix on the effect of various materials on concrete provides a ready reference for use by specification writers. The explanations and examples make this book much more suitable for textbook and reference purposes than the available handbooks on United States practice.

Iterational analysis of multistory frames with sidesway (Iterationsweise Berechnung von laengsverschieblichen Stockwerkrahmen)

Josef Eisenmann, *Der Bauingenieur* (Berlin), V. 31, No. 2, Feb. 1956, pp. 61-67
Reviewed by Aron L. Mirsky

Extension of author's paper in same journal, V. 28, No. 6, June 1953, pp. 198-199 ("Current Reviews," ACI Journal, Dec. 1953, *Proc.* V. 50, p. 335). Author states method is based on that of Hardy Cross, but represents a simplification thereof. Two worked examples are included: a one-bay, four-story symmetrical structure, and a two-bay, two-story unsymmetrical frame. Convergence is rapid in both cases.

For errata, see title source, Apr. 1956, p. 156.

Bending of sectorial plates under a concentrated load

Tsuyoshi Sekiya and Atsushi Saito, *Proceedings, Fourth Japan National Congress of Applied Mechanics* (Tokyo), 1954, pp. 195-198
Reviewed by Gunhard-Aestius Oravas

Sectorial plates under concentrated loads are solved by superposition of Washizu's integral equations. Various mixed boundary conditions are considered.

Cylindrical shell roofs formed by triangular lattice of rolled metallic members (Cubiertas laminares cilíndricas formadas por una malla triangular de perfiles metálicos)

Florencio del Pozo, *Bulletin* No. 176, Instituto Técnico de la Construcción y del Cemento, Madrid
Reviewed by E. Rosenblueth

The type of structure analyzed in this paper consists of a widely separated triangular steel lattice. All triangles are isosceles and their bases coincide with generatrices of cylindrical surfaces. The frames thus formed are covered with rectangular panels. Savings

are claimed for this type of roof over conventional concrete shells on the basis of light weight and elimination of form work. Considering membrane stresses as primary, complete solutions are found for secondary stresses under the assumption that the mesh is infinitesimal and that it behaves as a pronouncedly anisotropic elastic shell. Tabular arrangement simplifies computations, making design procedure adequate for routine office practice. Actual structures of this type have been built in Spain. The analysis is valid for similar arrangements of other materials, such as precast concrete members.

Tables for design of wall-type girders in reinforced concrete (in German)

O. F. Theimer, Wilhelm Ernst & Sohn, Berlin, 1956, 38 pp., 7.2 DM

Reviewed by J. J. Polivka

Basic analysis as has been developed and previously published by H. Bay, F. Dischinger, Craemer, R. Bortsch, K. Girkmann, A. Pücher, and others is thoroughly discussed, and the results presented in charts and tables which considerably simplify relatively complex calculations. Their usefulness is demonstrated in three examples.

Contribution to the analysis of a circular ring on elastic foundations (Beitrag zur Berechnung des Kreisringes auf elastischer Unterlage)

Gunhard Oravas, *Der Bauingenieur* (Berlin), V. 31, No. 5, May 1956, pp. 177-180

Reviewed by Aron L. Mirsky

Develops solutions for the circular ring on elastic supports with the torsional resistance of the support material taken into account. Several loading cases are investigated.

Materials

Reducing water content of slurry in wet-process cement manufacture

Johs. Rutle, *Pit and Quarry*, V. 49, No. 8, Feb. 1957, pp. 78-79, 82-83, 89

Author's Summary

Describes a method of reducing the water content of slurry by the use of a slurry additive, produced by a certain treatment of peat or boggy soil with alkali. In a mixture composed of peat (boggy soil) and alkali stored in air, complicated chemical reactions take place, resulting, among other things, in the formation of compounds rich in carboxyl groups, to which the water-reducing effect of the product is attributed.

For a certain slurry the water content can be reduced to 25 percent in plant operation. This additive acts as a grinding aid. The additive also reduces dust concentration in kiln exit gases. However, in kilns provided with certain heat-exchange devices, the additive may bring about a more irregular kiln operation.

Effect of the additive depends on many factors, such as the raw materials utilized for cement production and the nature of the peat or boggy soil. Use of the additive not only means less water in the slurry but also a slurry of another nature—a slurry which behaves in a different way in grinding, handling, and kiln operation.

Pavements

Road research—1956

Road Research Board, Department of Scientific and Industrial Research, London, 1957, 88 pp., $1.04 (Available from British Information Services, 45 Rockefeller Plaza, New York 20, N. Y.)

Summarizes road research in Great Britain for the past year. Principal topics of research are safety, traffic, materials (including soils, bituminous materials, and concrete), and methods of construction.

Salvaging old pavements by resurfacing

Bibliography No. 21, Highway Research Board, 1957, 48 pp.

An annotated bibliography including both rigid and flexible types of resurfacing. There are 175 references on rigid overlays published 1916-1956; 81 references on flexible overlays published 1932-1956. Each section is indexed.

No-joint concrete pavement laid in Pennsylvania

E. Mannix, *Contractors and Engineers*, V. 54, No. 7, July 1957, pp. 54-56

Covers design and paving operations on a 10,800-ft experimental strip of continuously reinforced jointless concrete pavement of two 24 ft wide pavements with a separation strip. The test strip was divided into slab thicknesses of 7, 8, and 9 in. compared to the customary 10-in. thickness used in Pennsylvania. Hard-grade bar mats were used as reinforcement.

Precast Concrete

Precast concrete yearbook—1957 (Betonstein Jahrbuch—1957)

Association of Precast Concrete Industries, Bonn, Germany, 1957, 527 pp.

A pocket-size handbook summarizing applications of precast concrete as long-span roofs, small plank-type floor and roof units, walls, structural frames, and pipe is presented in Section I, 370 pp. Design details and photographs are used to supplement the descriptions. Section II presents marketing information on the industry in Germany.

Spun concrete columns for housing in Bulgaria

S. Mirtschev, *Prefabrication* (London), V. 4, No. 45, July 1957, pp. 404-409

Gives manufacturing details and describes the erection of hollow precast concrete columns formed by the centrifugal spinning process. The columns are full height of the building, approximately 55 ft long, circular in cross section, and provided with a solid cross section in the vicinity of the beam seats. Their use in five-story apartment buildings is illustrated, including description of other precast concrete elements such as floor and wall panels and precast stairways to form the complete frame of the building.

Stability of prefabricated components during lifting operations (in German)

P. Csonka, *Bauplanung-Bautechnik* (Berlin,) V. 10, No. 9, Sept. 1956, pp. 355-360

Reviewed by I. M. Viest

Problems arising during the hoisting of prefabricated beams and slabs are discussed. Methods are given for locating the points of support so as to assure stable equilibrium during the lifting operations.

Properties of Concrete

Durability of reinforced concrete in buildings

Special Report No. 25, National Building Studies, Department of Scientific and Industrial Research (Britain), 1956, 30 pp.

Reports on condition of more than 50 reinforced concrete buildings in various stages of deterioration. Survey was limited to buildings, not including bridges, other structures, nor foundation works. Considerable attention was devoted to fire damage and its repair, particularly using the "cement gun" (shotcrete). The conclusions are what might be expected. Durability is achieved by using a good quality concrete as evidenced by a low water-cement ratio, well consolidated to avoid honeycomb, well designed to avoid cracks, and in sufficient dimensions as cover between reinforcement and the exposure to prevent entry of water.

Incidental conclusions are that the material is highly fire resistant and that the shotcrete method of repair is effective. An appended summary of the condition survey includes building subject to chemical attack, factories, granaries, warehouse buildings, and office buildings.

Fluorine gas makes concrete chemical-resistant

J. van der Eerde, *Rock Products*, V. 59, No. 8, Aug. 1956, pp. 263-266

Ceramic Abstracts Feb. 1957 (Barbour)

Describes briefly the process of combining a portion of the drying and curing period of concrete pipe and tile with treatment by SiF_4 developed from sodium silicofluoride in a tunnel under vacuum. Properties of the finished product are tabulated.

Shrinkage of concrete (in Danish)

Erik V. Meyer and Knud E. C. Nielsen, *Nordisk Betong* (Stockholm), V. 1, No. 2, 1957, pp. 129-153

Reviewed by Margaret Corbin

Shrinkage produces a number of effects which are decisive for the service of precast concrete units. These effects are: changes in dimensions, changes in shape, formation of cracks, and apparent or actual changes in the strength of materials. The mechanism of shrinkage in concrete is considered as a two-phase system consisting of cement paste and aggregate. Shrinkage is defined as a phenomenon represented by that volume change or tendency to volume change which accompanies the interaction between water and cement as well as the products of cement hydration. Accordingly shrinkage is primarily a local process in the cement paste, and the aggregate acts only as an inert filler, although it is a decisive factor in determining the effects produced by shrinkage in concrete. This concept of the shrinkage mechanism is used as a basis for drawing up a list of primary variables, whose qualitative effect on shrink-

age is discussed. A description is given of the ring method (Berthier) used for the experimental determination of shrinkage, primarily expressed in terms of cracking tendency exhibited by cement paste, mortar, and concrete.

Structural Research

Reduction of stress by alternating forces in the most highly compressed fibers of highly stressed concrete structures (La diminution de contrainte par les efforts alternes dans les fibres les plus comprimees des ouvrages en beton fortement sollicite)

A. Mogaray, *Annales des Ponts et Chaussees* (Paris), V. 126, No. 3, May-June 1956, pp. 281-316
Reviewed by Aron L. Mirsky

Author's thesis is that both non-instantaneous elastic deformations and nonelastic instantaneous deformations (as in elasto-plastic theories) must be used in accounting for the behavior of large structures; that alternating stresses (in particular, those due to hygrothermic cycles) can result in a reduction in total stress in highly-stressed concrete fibers, contrary to classical theory which calls for an increase; and that the principle of superposition of stresses is thus not valid in such cases, and its abandonment would lead to considerable economies. Experimental data obtained from vibrating wire gage readings on the Kitchener bridge over the Saone at Lyon are cited and discussed.

Research on the rigidity of a reinforced concrete beam (in Polish)

W. Kuczynski, *Inzynieria i Budownictwo* (Warsaw), No. 12, 1955, pp. 392-398
Polish Technical Abstracts No. 4 (24) 1956

Discusses the problem of determining the deflection of a beam of variable cross section subjected to different loads. Deals also with statistical research proposed for determining the best possible conditions for insuring accuracy of measurements of the deflection of reinforced concrete beams in laboratories or on building sites. On the basis of an analysis of the material collected, a hypothesis is formulated concerning a linear change in the rigidity of the cross section of a reinforced concrete beam $B = B_o - mM$ where: $B = EJ$—the rigidity of the cross section assumed to be a variable depending on the loading of the beam; $B_o = EJ$—original rigidity under slight loading; and M is the bending moment in a given cross section of the beam. Some 300 concrete beam and cylinder tests and the statistical material collected enabled conclusions to be drawn.

Strength of slender non-reinforced concrete walls (in Swedish)

Arne I. Johnson, *Nordisk Betong* (Stockholm), V. 1, No. 2, 1957, pp. 191-207
Reviewed by Margaret Corbin

It is desirable that thin concrete walls (height-to-thickness ratio $\geqq 25$) be used as load-bearing structural components in modern precast housing construction. The actual strength of such components was determined on 29 full-scale walls in Sweden and the results are here summarized. Walls were found to have a high load-bearing capacity, and this capacity did not decrease to any considerable extent until the slenderness ratio exceeded about 30. Tests made in Denmark by Glarbo (*Ingeniøren*, 1951) show that the decrease in the load-bearing capacity of walls subjected to eccentric loads is substantially greater in the case where the values of eccentricity at both supports are equal in magnitude and in direction than in the case where the eccentricity is confined to the load at one of the supports. Tests performed on walls provided with slots showed that the decrease in load-bearing capacity is small when the slots are located near a support.

General

Sixty years of hydro-electric development in Great Britain

J. Guthrie Brown, *The Structural Engineer* (London), V. 34, No. 11, Nov. 1956, pp. 373-403
Reviewed by C. P. Siess

General discussion with emphasis on changes in design and construction. Describes or mentions several concrete dams and tunnels.

Curing of concrete: 1925-1955

Bibliography No. 18, Highway Research Board, 1956, 98 pp., $1.80

This bibliography of 518 annotated references presents a comprehensive coverage of literature from the United States and Canada

on the curing of concrete. Much of the more important literature from the rest of the world is also included. One class of important publications is not well covered—bulletins and related publications from universities, governmental offices, and manufacturers.

The principal index of the bibliography is arranged by subject; each subject group is arranged alphabetically by author; and each author-subject group is arranged chronologically.

Backup materials as a source of efflorescence

JAMES E. YOUNG, *Journal of the American Ceramic Society*, V. 40, No. 7, July 1957, pp. 240-243

Reports results of an investigation to determine whether effloresecing salts can originate in concrete backup block. The investigation indicates that soluble salts can be transmitted from backup materials to be deposited upon the face of a brick wall, and further that the degree of efflorescence is related to the amount of soluble salts in the backup material.

Post-war developments in the coal mining industry

ANDREW YOUNG, HENRY A. LONGDEN, and BERNARD L. METCALF, *Proceedings*, Institution of Civil Engineers, London, V. 6 (Session 1956-57), Apr. 1957, pp. 662-695 (discussion, pp. 696-708)
Reviewed by ARON L. MIRSKY

Besides an interesting glimpse of a specialized branch of technology of which we usually think only in terms of product, prices and strikes, and accidents, paper includes some data (pp. 677-680; discussion, p. 700 especially) on concrete lining of shafts sunk using freezing of the ground.

Reason and use of structural types (Razón y ser de los tipos estructurales)

E. TORROJA, Instituto Tecnico de la Construccion y del Cemento, Madrid, 392 pp.

A philosophical discussion of structural systems tracing their origins, basic characteristics, and applications. Nontechnical in treatment, it is nevertheless of interest to architect, engineer, and layman alike. Discussed are classical building materials, wood and steel, and reinforced and prestressed concrete. Arches, vaults and domes, trusses, retaining walls and dams, roofs, slabs, bridges and aqueducts, functional formed structures, and unusual construction methods are considered. The remainder of this well-illustrated and entertaining book is devoted to esthetic expression and forms, their history, design considerations, and organization.

Recent trends in the design of industrial buildings

A. V. HOOKER, *The Structural Engineer* (London), V. 34, No. 8, Aug. 1956, pp. 271-283
Reviewed by C. P. SIESS

Rather general discussion covering concrete, steel, timber and aluminum structures. Mentions precast, prestressed, and prepacked concrete.

Note on the history of reinforced concrete in buildings

S. B. HAMILTON, *Special Report* No. 24, National Building Studies, Department of Scientific and Industrial Research (Britain), 1956, 30 pp.

Traces the advance of ideas on reinforced concrete construction from a beginning in the middle of the 19th century through its development in Europe to its introduction in Britain at about the end of the century. From that point traces British practice applied to reinforced concrete work in buildings.

Compression testing machines for concrete

P. J. F. WRIGHT, *The Engineer* (London), V. 203, No. 5283, Apr. 26, 1957, pp. 639-641
Reviewed by ARON L. MIRSKY

Tests on nine different testing machines, using concrete cubes produced under carefully controlled conditions, showed some machines gave consistently lower crushing strengths. The following causes of these discrepancies were found: misalignment of testing-machine components, lack of planeness of the platens, failure to maintain a constant loading rate to failure, and movement of self-aligning platens under high loads. These results are embodied in five recommendations for testing machines.

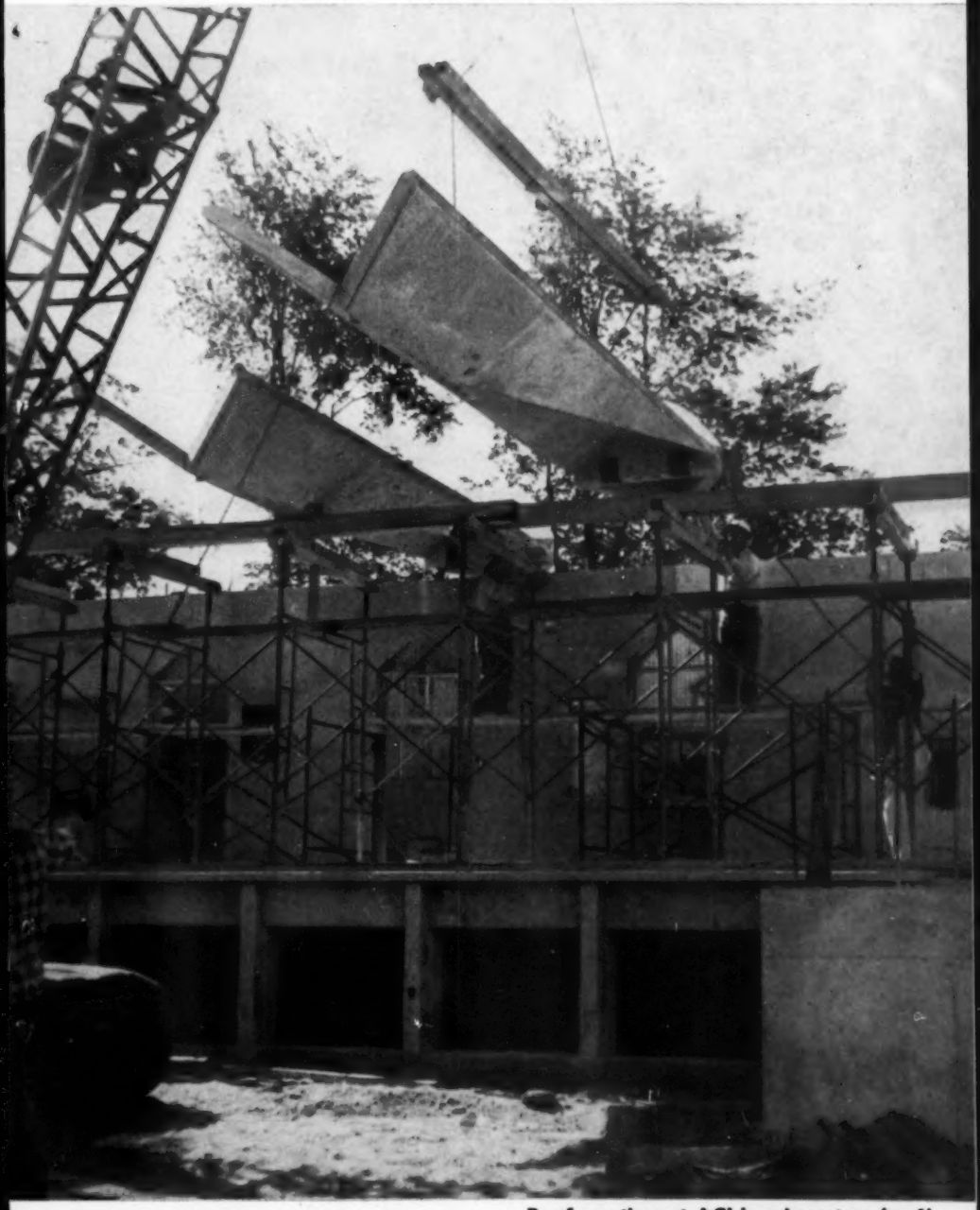
Reinforced concrete ships and barges —a selected bibliography

Indian Concrete Journal (Bombay), V. 31, No. 4, Apr. 1957, p. 127

Presents a bibliography on the use of reinforced concrete for ships dating from 1918 to 1956, confined entirely to English language publications.

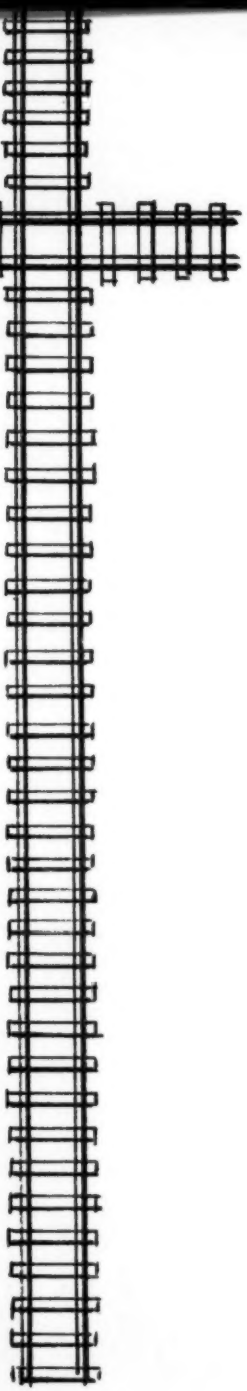
ACI News Letter

Vol. 29, No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE October 1957



Roof erection at ACI headquarters (p. 6)

guide to Intrusion-Prepakt maintenance services:



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ACI

News Letter Contents

October 1957

On the Cover—Lifting roof sections for placement on the new ACI headquarters building. This phase of construction has now been completed. The roof system, cantilevered front and rear from interior corridor walls of concrete, will project to shade exterior glass walls.

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1958 Nominees Announced

President—D. McHenry
Vice-president—J. W. Kelly

The ACI Committee on Nominations, headed by T. Y. Lin, University of California, has announced names of candidates to be submitted to member ballot at the 54th annual convention, Feb. 25-27, at the Morrison Hotel, Chicago.

For President

Douglas McHenry, director of development, Portland Cement Association, Chicago, has been nominated to succeed Walter H. Price as president for a 1-year term beginning at the February, 1958 convention. Mr. McHenry has been a member of ACI since 1936. He is now completing his second year as Institute vice-president. Mr. McHenry is well known in engineering circles as the author of numerous articles on concrete technology and, in particular, for his work while head of the structural research section of the engineering laboratories of the U. S. Bureau of Reclamation prior to joining PCA in 1952.

McHenry

Mr. McHenry attended the University of Minnesota from 1921 to 1924 and then joined the Southern California Edison Co., working on the Florence Lake and Shaver dams. Before joining the staff of the Bureau of Reclamation in 1940 he served for 6 years with the Tennessee Valley Authority on construction of Norris and Hiwassee dams and power plants, and in research studies of the structural behavior of TVA works.

Active in Institute committee work for many years and a frequent contributor of technical papers for the JOURNAL, he is now a member of the Standards Committee, Committee 209, Volume Changes and Plastic Flow in Concrete, and Committee 314, Rigid Frame Bridges. He has served as chairman of the TAC and Committee 209, and was a member of the Board of Direction in 1955.

For vice-president

J. W. Kelly, professor and vice-chairman, department of civil engineering, University of California, Berkeley, has been nominated for a 2-year term as vice-president.

Professor Kelly graduated from Purdue University in 1921. For 2 years he engaged in

J. W. Kelly

waterworks engineering. Then, after a brief term as assistant in the testing materials laboratory, Purdue University, he joined the staff of the Portland Cement Association where he remained for 7 years. Returning to Purdue University as a concrete specialist of the engineering extension department, he prepared a practical booklet on concrete making for the small user of cement. In 1931 he joined the University of California staff.

Professor Kelly has much work to his credit in his 30 years of ACI membership. He was awarded the Wason medal in 1934 for the paper, "Cement Investigations for Boulder Dam with the Results up to the Age of One Year," prepared in collaboration with Raymond E. Davis, R. W. Carlson, and G. E. Troxell. In 1946 he received the ACI Construction Practice Award with Bernerd D. Keatts for the paper "Two Special Methods of Restoring and Strengthening Masonry Structures." Professor Kelly served on the ACI Board of Direction from 1952-54, was elected in 1956 to his present term as director; he is a member of ACI Committee 611, Inspection of Concrete, and formerly served as chairman of that committee.

Phil M. Ferguson, chairman of the civil engineering department, University of Texas, Austin, was elected to a 2-year term as vice-president at the 1957 convention, and will continue in that office.

For director, four nominees

Nominated to the four Board of Direction vacancies for 3-year terms beginning at the February, 1958 convention are: A. Amirikian, S. J. Chamberlin, Bryant Mather, and Cedric Willson.

Mr. Amirikian, chief design engineer, Bureau of Yards and Docks, U. S. Navy, Washington, D. C., graduated from Cornell University in 1923. He spent 5 years with private engineering and fabricating firms before joining the Bureau of Yards and Docks in 1928.

In 1945 he became a member of ACI and has served on numerous committees, and has had several papers on precast concrete published in the JOURNAL.

A large segment of Mr. Amirikian's work in recent years has been in the field of thin-shell precast concrete construction and design of protective structures against conventional bombs and atomic blast. He has been widely honored in recent years for his outstanding contributions to the development of reinforced concrete.

He presently is chairman of ACI Committee 324, Precast Reinforced Concrete, Thin Sections, and a member of ACI Committee 314, Rigid Frame Bridges, and the joint ACI-ASCE Committee 323, Prestressed Reinforced Concrete.

S. J. Chamberlin, professor of theoretical and applied mechanics, Iowa State College, Ames, graduated from the University of Illinois in 1928 and received his MS, majoring in highway and railway engineering, from Iowa State in 1931.

Professor Chamberlin's experience includes work with the Missouri Pacific Railroad and the Illinois Division of Highways. For several years he taught drawing at the Burlington Junior College, and joined the faculty of Iowa

State College in 1936. His work includes both teaching and research.

Professor Chamberlin has been an ACI member since 1940. He has contributed several papers for publication in the ACI JOURNAL and in 1939 was awarded the Wason Medal for the paper "The Bond Between Concrete and Steel," which he prepared in collaboration with Herbert J. Gilkey and R. W. Beal. He has been active in committee work for many years. Long the chairman of Committee 115, Research, he is now secretary of that group. Professor Chamberlin is also a member of Committee 711, Precast Floor Systems for Houses, and Committee 208, Bond Stress.

Bryant Mather, chief of the Special Investigations Branch, Concrete Division, Waterways Experiment Station, Corps of Engineers, U. S. Army, Jackson, Miss., received his degree in geology from Johns Hopkins University in 1936. He continued with graduate studies there and at American University, Washington, D. C., until 1941. He then joined the Corps of Engineers in concrete research.

An Institute member since 1944, Mr. Mather has served three terms as chairman of the ACI Technical Activities Committee. He has been chairman of several subcommittees of ASTM Committee C-9, including Admixtures for Concrete, and Chemical Reactions in Concrete. At present he is a member of TAC as well as ACI Committee 116, Nomenclature, Committee 612, Recommended Practice for Curing Concrete, and Committee 212, Admixtures.

Cedric Willson, vice-president and chief engineer, Texas Industries, Inc., Fort Worth, graduated from the University of Kansas in 1926. For the two following decades he was engaged primarily in research on special cements, production methods, and plant operation. Since 1948 his work has been with the production and use of Haydite lightweight aggregate, including structural concrete, lightweight concrete block, and reinforced concrete pipe.

Mr. Willson is the author of several technical papers on concrete and concrete products published by ACI, National Concrete Masonry Association, and American Institute of Mining and Metallurgical Engineers. He served as project consultant, concrete masonry construction, at Yucca Flat "Operation Cue" atomic test, May 1955. He is director and chairman of the Technical Problems Committee of National Concrete Masonry Association and a member of ASTM Committee C-9.

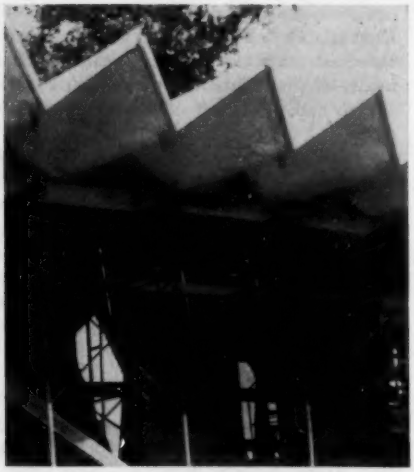
Active in ACI since joining in 1952, he is now a member of Committee 213, Properties of Lightweight Aggregates and Lightweight-Aggregate Concrete; Committee 331, Structures of Concrete Masonry Units; and Committee 613, Recommended Practice for Proportioning Concrete Mixes. He was local chairman for the 1957 convention in Dallas.

Nominating committee

Members of the Nominating Committee who prepared the new slate of officers are, in addition to Chairman Lin: Boyd G. Anderson, Ammann and Whitney, New York; Herbert K. Cook, Master Builders Co., Cleveland; Roger Corbetta, Corbetta Construction Co., Inc., New York; and Thor Germundsson, Portland Cement Association, Chicago. Past-president members of the committee are: Frank Kerekes, Michigan College of Mining and Technology, Houghton; Charles H. Scholer, Kansas State College, Manhattan; and Charles S. Whitney, Amman and Whitney, New York and Milwaukee.

Twenty candidates have been chosen for the 1958 Nominating Committee, five of whom will be elected at the February convention. Candidates are:

WILLIAM C. E. BECKER
WALTER E. BLESSEY
DELMAR BLOEM
FRED BURGGRAF
T. F. COLLIER
R. E. COPELAND
JAMES N. DE SERIO
WILLIAM J. ENEY
GEORGE C. ERNST
E. A. FINNEY
BEN C. GERWICK, JR.
GEORGE E. LARGE
HENRY LAYNE
JAMES A. MCCARTHY
JAMES MICHALOS
RICHARD MIELENZ
GERALD MILSOM
J. P. THOMPSON
GEORGE W. WASHA
C. A. WILLSON

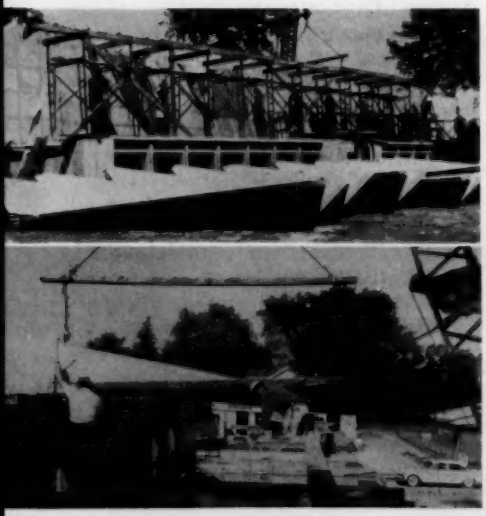


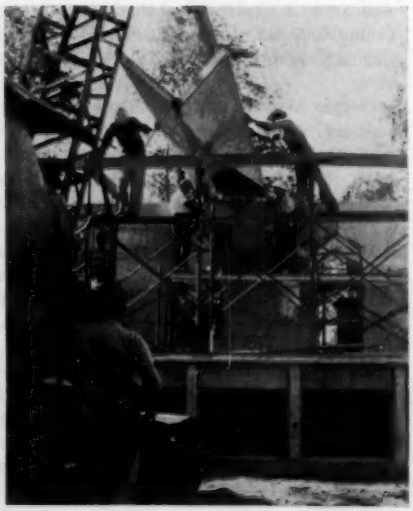
New home of its own in Detroit for ACI, designed by Minoru Yamasaki of Yamasaki, Leinweber and Associates, Royal Oak, Mich., has been roofed with folded plates of precast concrete. On-the-job precasting was described in the September "News Letter."

●

Safeway adjustable scaffolding (left) must remain in place until the roof sections have been joined all along the center of the building. Pictures below show sequence of lifting operations. Slabs were first placed on the ground near their approximate position in the building, then hoisted individually to rest on temporary scaffolding

ACI erects folded roof

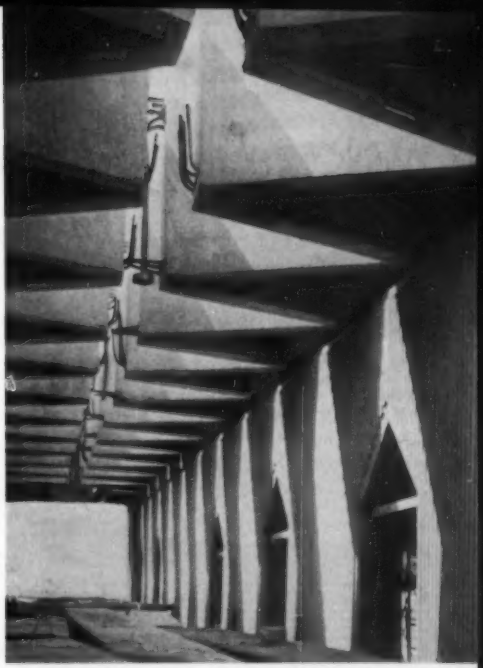




Building design calls for cantilevered roof supported on central corridor walls of cast-in-place concrete; exterior walls will be almost entirely of glass. At building's ends a decorative, roof-high screen of precast concrete units will shield the glass wall panels.

●

Encircling the building—another ornamental element in precast concrete—the garden wall will contrast in design with the end wall screen. The Institute expects to occupy its new headquarters early in 1958. General contractor for the work is Pulte-Strang, Ferndale, Mich.

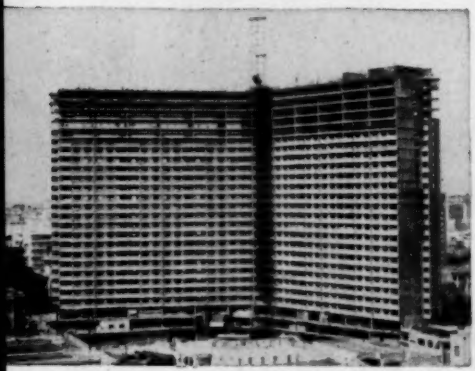


Patterns of light and shadow in this central corridor will remain much the same when building is complete, since skylights pierce the roof over the hallway. Here are clearly shown the triangular beam boxes at the corridor ends of folded-plate roof members, which will be joined to form X-beams. Fluted wall texture produced by special form lining

sections

General Hospital Neurological Building, Philadelphia; Archt. — Harbeson, Hough, Livingston & Larson; Contr. — McCloskey & Co., Inc.; Pozzolith Ready-Mixed Concrete — The Warner Co. — All of Philadelphia.

Prudential Insurance Bldg., Jacksonville, Fla.; Archt. — Kemp, Bunch & Jackson; Engr. — Reynolds, Smith & Hills; Contr. — Daniel Construction Co.; Pozzolith Ready-Mixed Concrete — Capitol Concrete Co. — all of Jacksonville.



"17 y M" — Reinforced concrete apartment building, Havana, Cuba; Archt. — Ernesto Gomez Sampera and Martin Dominguez; Struc. Engr. — Saenz-Cancio-Martin; In Charge of Construction: Bartolome Bestard and Manuel Padron.

Capitol Records Office Building, Hollywood, Calif.; Archt. Engr. — Welton Becket, F.A.I.A., and Assocs., L. A.; Struc. Engr. — Murray Erick Assocs., L. A.; Gen. Contr. — C. L. Peck, L. A.; Pozzolith Ready-Mixed Concrete — Transit-Mixed Concrete Co., Pasadena.

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The three controls provided by Pozzolith are lowest water content, control of entrained air and control of rate of hardening . . . vital factors in obtaining uniform, better quality concrete.

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ACI Regional Meeting

Seattle—November 5-6

With plans completed for the 10th Regional Meeting of the American Concrete Institute in Seattle, R. H. Lochow, chairman of the local committee, looks forward to a well attended, informative, and interesting session.

The two-day program November 5 and 6 at the Benjamin Franklin Hotel will emphasize the recent concrete projects and techniques in the Northwest.

The technical program will include such subjects as concrete for radiation shielding, shear walls, plastic flow characteristics of lightweight-aggregate concrete, and reinforced masonry construction. Other papers will report on warping due to shrinkage, prestressed cylindrical shells, prestressed lift slabs, precast colored panels, and precast and prestressed construction. Complete program of papers scheduled for presentation at the meeting appeared in the September ACI "News Letter."

Inspection trip scheduled for November 6 includes the Boeing Developmental Center and St. Edward's Church, a nationally-recognized precast concrete job completed about a year ago. On Thursday, November 7, after the formal portion of the meeting is concluded, plans are being made to visit several dams and concrete projects somewhat more remote from Seattle.

The lighter side of the meeting includes a social hour and a banquet scheduled for Tuesday evening at the Washington Hotel; a highly entertaining ladies' program includes two luncheons, a fashion show, and Grayline tour of Seattle, as well as the Tuesday evening affair.

Following are the members of the local committee who have assisted Mr. Lochow in making arrangements for the regional meeting: Roscoe C. Hildebrandt, PCA, Seattle, publicity; Fred H. Rhodes, Jr., University of Washington, registration and hospitality; Alfred L. Miller, University of Washington, education contact; Ray A. Young, Darex AEA Division, Charles R. Watts Co., finance; Peter H. Hostmark, structural engineer, technical program; William P. Jackson, Glacier Sand & Gravel Co., inspection trips; Leland L. Sphar, Concrete Products Association of Washington, ladies entertainment; Abbott W. LaGrelius, banquet, and Donald G. Metcalf, hotel reservations and luncheons.

Reservations may be made directly to the Benjam Franklin Hotel, Seattle.

ACI technical committee appointments

Listed below are committee members recently appointed to several of the Institute's technical committees. Included are *new* appointments only.

Committee 116, Nomenclature

D. L. Bloem
National Ready Mixed Concrete Association and National Sand and Gravel Association
Washington, D. C.

J. R. Dise
National Bureau of Standards
Washington, D. C.

J. E. Gray
National Crushed Stone Association
Washington, D. C.

Bryant Mather
Waterways Experiment Station
Corps of Engineers
Jackson, Miss.

W. J. McCoy
Lehigh Portland Cement Co.
Coplay, Pa.

Mark Morris
Iowa State Highway Commission
Ames, Iowa

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USE SONOCO SONOVOID® FIBRE TUBES to form voids in concrete roof slabs!

Middleville Road High School, Northport, Long Island, N. Y. Ellis Chinges Construction Corp., general contractors. Severud, Elstad & Krueger, structural engineers. Ketcham, Gina & Sharp, architects

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Sonoco SONOVOID® Fibre Tubes

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The circular concrete roof slab for library building on the campus of the Middleville High School weighs less because it contains Sonoco SONOVOID® Fibre Tubes!

This cast-in-place slab requires less concrete and less reinforcing steel because Sonoco SONOVOID® Fibre Tubes displace low-working concrete at the neutral axis.

Voids in concrete construction save materials without impairing structural strength!

Easy to handle, low-cost Sonoco SONOVOID® Fibre Tubes are specifically developed for use in bridge decks, wall, floor, roof and lift slabs; also in concrete piles. For precast units or units cast in place.

Order in sizes from 2.25" O.D. to 36.9" O.D. up to 48' long. Specify lengths to meet your needs or saw to size on the job. End closures available.

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Walter H. Price
U. S. Bureau of Reclamation
Denver, Colo.

C. A. Sirrine
Concrete Products Association of Michigan
Detroit, Mich.

Committee 201—Durability of Concrete in Service

Robert F. Adams
U. S. Bureau of Reclamation
Denver, Colo.

Committee 212—Admixtures

John G. Dempsey
Compania Anonima de Concreto
Bachaquero, Estado Zulia, Venezuela

Committee 213, Properties of Lightweight Aggregates and Lightweight-Aggregate Concrete

Otto C. Frei
Georgia Lightweight Aggregate Co.
Atlanta, Ga.

H. I. King
Cooksville Co.
Toronto, Ont., Canada

Morton Sherman
Zonolite Co.
Chicago, Ill.

Committee 314, Rigid Frame Bridges

Narbey Khachaturian
University of Illinois
Urbana, Ill.

Committee—323—Prestressed Reinforced Concrete—Joint ACI-ASCE

Morris Schupack
The Preload Co.
New York, N. Y.

Committee 334—Concrete Shell Structures

Boyd G. Anderson
Ammann and Whitney
New York, N. Y.

Felix Candela
Cubiertas Ala, S. A.
Mexico City, Mexico

Committee 401—Specifications for Structural Concrete

R. R. Matheu
Hall, Pregnoff & Matheu
San Francisco, Calif.

Committee 605—Hot Weather Concreting

George Otterson
Corps of Engineers
Washington, D. C.

John J. White
Parsons-Brinckerhoff-Hall & MacDonald
New York, N. Y.

Committee 714—Concrete Bins and Silos

Harold T. Barr
Louisiana State University
Baton Rouge, La.

Joint committee on limit design organized

Alfred L. Parme, Portland Cement Association, Chicago, is chairman of ACI-ASCE Joint Committee 328, Limit Design, organized in 1956 to develop recommended practice applicable to limit design as applied to reinforced concrete.

Members of the committee are:

Richard R. Bradshaw
Richard R. Bradshaw, Inc.
Van Nuys, Calif.

Edward Cohen
Ammann & Whitney
New York, N. Y.

Freeman Drew
Elmhurst, Ill.

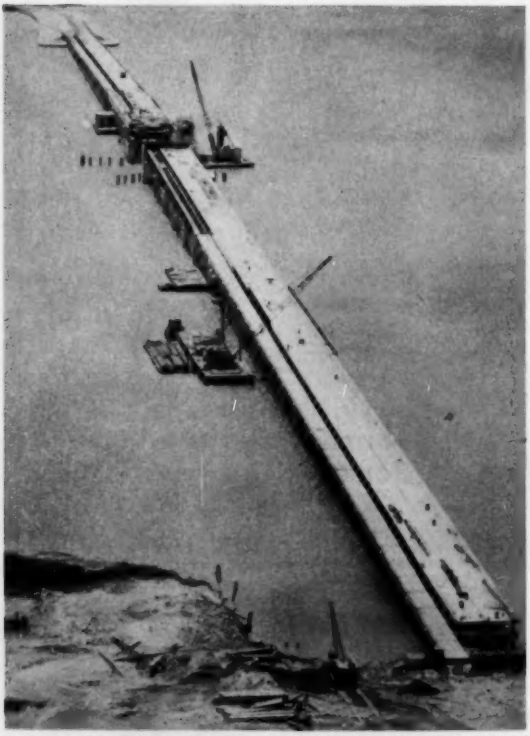
Eivind Hognestad
Portland Cement Association
Chicago, Ill.

Newlin D. Morgan, Jr.
University of Wyoming
Laramie, Wyo.

Louis A. North
Allabach & Rennis, Inc.
Philadelphia, Pa.

Continued on p. 14

Why they used PLASTIMENT in the PELICAN ISLAND CAUSEWAY



The Pelican Island Causeway at Galveston, Texas carries a two-lane highway and a railroad track across more than a mile of water. Earth fill supported by cantilevered concrete sheet piles runs 1000 feet from the mainland. Concrete trestle construction totaling 4500 feet comprises the major portion of the crossing. Nine steel girder spans are used in the approach to the 160-foot single leaf bascule which spans the ship channel.

OWNER: Galveston County Navigation District No. 1
CONSULTING ENGINEERS: Parsons, Brinckerhoff, Hall and Macdonald (New York City). Resident Engineer, Arthur Alpert
GENERAL CONTRACTORS: Farnsworth & Chambers (Houston), Kansas City Bridge Co. (Kansas City), Texas Gulf Construction Co.(Galveston).
PRESTRESS SUB-CONTRACTOR: Span, Inc. (Dallas).
READY-MIX SUPPLIERS: Horton and Horton (Houston), W. A. Kelso (Galveston).

- prestressed beams
- prestressed bearing piles
- prestressed railroad decks
- tremie concrete

Rapid production and erection of prestressed bearing piles, beams, and railroad decks are among the factors responsible for outstanding success in construction of the Pelican Island Causeway.

Two properties of Plastiment were deemed extremely advantageous to the concrete work on this project. The first of these was the increased density of the finished product, which was vital to resist spalling and other damage from the polluted sea water in Galveston Bay. The second advantage was the retarding property of Plastiment, which delayed the initial set of concrete in the 400-foot prestressing bed on a hot summer's day long enough to complete the pour, and start the steam curing before temperature cracks appeared. Additional advantages of better workability with less water and increased strength were apparent during production.

Plastiment Concrete Densifier is a versatile aid for achieving a uniform, dense concrete at all seasons of the year. Uniform quality means economy! Let Plastiment help you... write or call for information.

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Positions and Projects

ACI releases 1957 Detailing Manual

The latest edition of *Manual of Standard Practice for Detailing Reinforced Concrete Structures* (*ACI 315-57*) is now available from ACI headquarters.

In the past, separate manuals for the detailing of reinforced concrete building structures and reinforced concrete highway structures have been published because of the slight differences in procedure in these two fields. The 1957 edition includes both types of construction in one volume.

The primary use of the Detailing Manual is to present improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel in reinforced concrete structures. The use of these improved methods and standards not only results in better reinforced concrete construction, but also simplifies and reduces the amount of work ordinarily required to prepare such drawings.

Although the purpose of this manual is to instruct bar detailers how to detail, it serves the secondary purpose of suggesting to architects and engineers how best to convey their design requirements to the detailer. A chapter has been included to give a brief résumé of what information should be conveyed by the designer to the detailer and how to accomplish this in the simplest and clearest manner. Price of the manual is $4.00 to nonmembers, $2.50 to ACI members.

Louis S. Wertz retires

Long-time ACI member Louis S. Wertz recently retired as president of Intrusion Prepakt, Inc., Cleveland. A recognized authority on masonry and concrete construction, Mr. Wertz organized Intrusion-Prepakt in 1934; the concern was originally known as the Durite Co. Mr. Wertz has also retired as head of The Prepakt Concrete Co., and Concrete Chemicals Co., also of Cleveland.

Mr. Wertz is a member of the International Congress of Large Dams and the American Railway Engineering Association, in addition to holding membership in ACI.

NCCA to hold first annual convention

The first annual convention of the National Concrete Contractors Association and the 1958 Concrete Exposition will be held in Chicago, Feb. 24-26, at the Chicago Coliseum.

James D. St. Clair is executive secretary of the association which has been established for contractors engaged in placing concrete throughout the country.

BRI announces adhesives and sealants conference

The Building Research Institute will conduct a major research conference on "Adhesives and Sealants in Building" on Dec. 4-5, 1957, at the Shoreham Hotel, Washington, D. C.

William H. Scheick, executive director of BRI, announces that this is the first conference of its kind to emphasize the rapidly growing importance of these materials in present-day construction. Every kind of adhesive and sealant material will be identified, the nature of its use explained, and specific applications reported on at this meeting according to Mr. Scheick.

Among the technical papers presented will be "Bonding Agents for Cementitious Materials," by C. Richard MacLellan, Larsen Products Corp., covering the development of adhesives for bonding cements and plasters to masonry and concrete surfaces.

Richmond Screw Anchor Co. ownership changes

Majority control of the Richmond Anchor Screw Co., Brooklyn, N. Y., manufacturer of concrete form tying and anchoring devices, has been acquired by the Shattuck Denn Mining Corp., according to announcement by Charles A. Snyder, president of the Richmond Co.

Shattuck Denn is a leading producer of nonferrous metals, and operates zinc, lead, gold, and silver mines at Humboldt, Ariz., and a uranium mine in Uravan, Colo.

Joint committee

Continued from p. 11

William Prager
Brown University
Providence, R. I.

Emilio Rosenblueth
Consulting Engineer
Mexico City, Mexico

Herbert Sawyer
University of Connecticut
Storrs, Conn.

Edward E. Walters
Consulting Engineer
Dallas, Tex.

C. W. Yu
Portland Cement Association
Skokie, Ill.

Dewey and Almy announce appointments

Dewey and Almy Chemical Co., Cambridge, Mass., has announced the appointment of Walter Adler, Jr., as product manager for WRDA, new water-reducing agent for concrete. Mr. Adler, a 1948 graduate of Harvard College, was formerly product manager for industrial gasketing compounds in the firm's container products department.

Ralph G. Crimm, Jr., has joined the firm as sales engineer, specializing in sales of WRDA. He has had extensive experience in the construction field as a salesman and concrete technician.

Richard G. Allen has been appointed sales engineer for construction specialty products in the Western Division, Dewey and Almy Chemical Co. His appointment was announced by Sam Burks, western construction specialties manager, who said Mr. Allen will work from Western Division headquarters at San Leandro, Calif.

A civil engineering graduate from the Illinois Institute of Technology, Mr. Allen was previously associated with the Universal Form Clamp Co. and Sonneborn, Inc.

Tung Au joins Carnegie Tech staff

Tung Au, formerly assistant professor of engineering mechanics at the University of Detroit, is now visiting associate professor in the civil engineering department at Carnegie Institute of Technology, Pittsburgh.

R. J. O'Heir named by Perlite Institute

Richard J. O'Heir has been appointed secretary-treasurer of the Perlite Institute, New York. Mr. O'Heir, who formerly served as technical director, supervised the institute's research and development projects for the past 5 years. He is now responsible for policy and administrative decisions. He has been an ACI member since 1955.

Hinds made Master Builders vice-president

Appointment of Frank Hinds as vice-president in charge of United States sales of The Master Builders Co., Cleveland, was recently announced by S. W. Flesheim, chairman and E. L. McFalls, president.

Hinds, who joined Master Builders in 1932, is a registered professional engineer, member of the American Concrete Institute and the American Ordnance Association.

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Henry L. Kennedy

Henry L. Kennedy, past president of the American Concrete Institute, died Sept. 10, 1957, just as his most recent efforts on behalf of the Institute—in planning and financing a new ACI headquarters—neared fruition. Mr. Kennedy, sales manager of the construction specialties department of Dewey and Almy Chemical Co., Cambridge, Mass., was an internationally known authority on concrete. He pioneered the use of grinding aids for the efficient manufacture of portland cement, and air-entraining agents to improve concrete durability.

Henry L. Kennedy

An ACI member for the past 23 years, Mr. Kennedy contributed frequent papers for publication in the JOURNAL and devoted much time and effort to committee and administrative work. He was president of the Institute in 1953, and past-president member of the Board of Direction in 1954-56. He had served as Institute vice-president in 1951-52, and was a director 1944-49. He headed the ACI Building Committee, leading in the planning and fund raising activity for the ACI headquarters building whose construction now nears completion in Detroit. At the time of his death, he was also a member of ACI Committee 115, Research, and Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Born in Medfield, Mass., Aug. 15, 1897, Mr. Kennedy studied architectural engineering at Northeastern University, and architectural construction at Wentworth Institute, and did graduate work at Beaune University in France. A veteran of World War I, after several years as an architect and engineer, he joined Dewey and Almy in 1929 to organize its construction specialties department.

He also lectured on structural design at the Franklin Institute from 1924-28; headed a construction course at Wentworth Institute's evening school from 1924-34, and aided in organizing and directing the course in concrete technology at the Harvard University's graduate school of engineering. Recipient of the Herschel Award of the Boston Scoiety of Civil Engineers, Mr. Kennedy was co-author with Robert F. Blanks of *The Technology of Cement and Concrete. V. I—Concrete Materials*, published in 1955.

He had been active in several committees of the American Society for Testing Materials, and a member of the Boston Society of Civil Engineers, the American Institute of Architects, and the Society of American Military Engineers.

Frederick W. Kellberg

Frederick W. Kellberg, 58, civil engineer, died recently in Oakland, Calif. Mr. Kellberg was senior partner in the firm of Kellberg, Paquette & Maurer, San Francisco, consulting engineers.

A graduate of the University of California, he had been in private practice in San Francisco since 1929. He was structural consultant on such projects as the Cow Palace, the Equitable Life Assurance Society Building, and the State Public Health Building in Berkeley, Calif. He had been an ACI member since 1953.

Building committee report

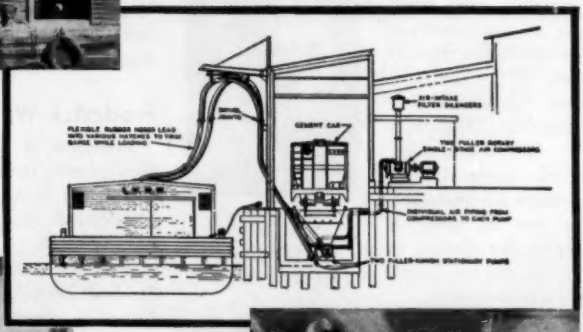
as of August 31, 1957

ACI Building Committee shows the following contributions to the ACI Building Fund by categories as of Aug. 31, 1957:

Cement producers	$ 69,512.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,176.22
Contractors	18,445.00
Ready-mixed concrete and aggregate industry	11,033.87
Admixtures manufacturers	7,458.93
Membership at large	5,091.74
Concrete machinery and specialty products	3,585.00
Concrete products manufacturers	2,520.00
Eastern Canada	1,496.90
Special contributions	1,209.08
Total	$194,528.74
Pledges in hand	7,300.00
GRAND TOTAL	$201,828.74

Hoses being placed in hatches of barge. Cement car in shed over pumps in pit.

Fuller-Kinyon System unloads cars . . . conveys to barges . . . unloads barges for Lehigh Valley Railroad



Unloader behind bulkhead in end of barge.

FULLER COMPANY GENERAL AMERICAN

Fuller-Kinyon Pumps in pit underneath car.

Bulk-cement handling by the Lehigh Valley Railroad at its Jersey City terminal is speeded up by the Fuller-Kinyon System. Fuller-Kinyon Pumps convey from cars to barge and Fuller-Kinyon Remote-Control Unloaders convey from barge to storage at various points in New York Harbor . . . speedy, efficient, time-saving handling from time cars arrive until final delivery to consumer.

Two stationary pumps in pit underneath car, convey through two lines to barge at rate of 900 to 1000 bbl. an hour. Unloading barge is accomplished at rate of 150 bbl. an hour. One man operates and controls all movements of the Unloader by a flexible-electrical cable and hand-control switch, wherever cement is to be picked up in the barge.

Air for the two stationary pumps is supplied by two Fuller Rotary Compressors, installed on the pier adjacent to the car-unloading station, at 614 c.f.m., 12-lb. pressure. Air for the Unloader is furnished by consumer's plant at destination.

Write for Bulletin FK-27, illustrating and describing this very interesting plant operation.

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GENERAL AMERICAN TRANSPORTATION CORPORATION SUBSIDIARY

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P-159
1482

St. Louis runway overlay will be 8-10-in. concrete

The city of St. Louis has awarded a contract for concrete overlay on the major portion of a 6000-ft concrete runway at Lambert-St. Louis Municipal Airport. The improvement is designed to accommodate a 100,000-lb dual wheel load, the standard specification for intercontinental type airports. Contract with Israel Brothers, Inc., Clayton, Mo., calls for completion within 90 days; urgency is due to the fact that the runway is the only one used for instrument landings at the field.

The airstrip to be strengthened is the northeast-southwest runway. A central portion of the concrete strip, 4700 ft long, and 7 in. thick, was constructed in 1941. Ends of the runway, built in 1943, are of 8-in. concrete. The 7-in. slab is still in good condition, although the 8-in. pavement shows evidence of structural failure at the northeast end because the greatest number of takeoffs and landings occur there.

The city plans to extend the runway in the future, but since a large part of it is still in good condition it was considered economical to cover the strip with an overlay now, rather than wait until serious deterioration takes place.

The 7-in. pavement will be covered with 8-in. concrete, and the 8-in. pavement at the northeast end will be covered with a 10-in. overlay because of the heavier loadings there. No work will be done at this time on the southwest end. The pavement will be removed where the runway intersects a 10,000-ft northwest-southeast strip, the longitudinal runway grade will be adjusted, and a new 14-in. pavement built.

Horner & Shifrin, St. Louis, are consulting engineers for the project, which calls for 77,000 sq yd of 8-in. pavement, 17,800 sq yd of 10-in. pavement, and 18,200 sq yd of 14-in. pavement.

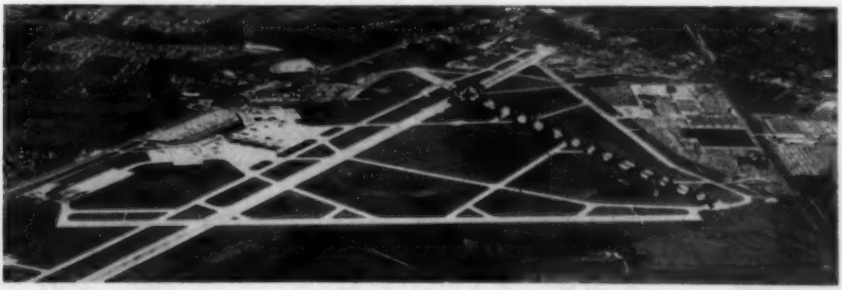
Schaffner becomes associate dean

Charles E. Schaffner has been appointed associate dean of Polytechnic Institute of Brooklyn. Dean Schaffner was formerly professor of civil engineering and director of the evening session, the largest accredited evening school of science and engineering in the United States.

A civil engineering alumnus of The Cooper Union, he continued his engineering education at the University of Illinois, New York University and the Polytechnic Institute of Brooklyn. Dean Schaffner came to Polytechnic in 1946 as assistant professor and head of the highway materials testing laboratory. He has been an ACI member since 1953.

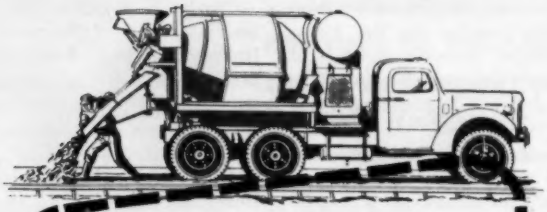
Thurber opens engineering office

R. C. Thurber, formerly materials engineer in the British Columbia Department of Highways, Victoria, B. C., has joined with two associates to form a consulting practice in Victoria. The firm intends to make consulting work on concrete a major activity; a research and development laboratory will be operated.



The 6000-ft instrument runway at Lambert–St. Louis Municipal Airport (indicated by crosses in photo) soon will be covered by concrete overlay 8-10 in. thick. The historic airfield, which served as home base for Charles Lindbergh in the 1920's, is now used commercially and as an industrial test field for McDonnell Aircraft Corp. jet planes

No time lost when concrete is



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AGGREGATE STOCKPILE—Five sizes of aggregate are being brought by rail to their siding at the construction site of the Cochrane hydroelectric project on the Missouri River at Great Falls, Mont. At each of two unloading stations, a 30- x 8-ft apron feeder under the receiving hopper transfers material to 75-ft conveyor which brings the aggregate up to ground level. There a 30-in. x 100-ft self-propelled, radial stacking conveyor, which can pivot 180 deg on its receiving end, delivers material to the desired location. Two identical stockpiling systems, one for sand and gravel (foreground of photograph), the other for aggregate up to 6 in., were designed and built by Pioneer Engineering, Division of Poor & Co., Minneapolis.

Winston-Green Contractors, a joint venture firm of Winston Brothers Co., Minneapolis, and Green Construction Co., Des Moines, Iowa, will build the dam and powerhouse for the Cochrane project. During peak concreting operations they anticipate rail delivery of some 1500 tons of aggregate per day to be stockpiled at the northwestern job site

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Fuller acquires Dracco

The Fuller Co., Catasauqua, Pa., has directly entered the field of dust collection and recovery by the purchase of Dracco Corp. of Cleveland. This acquisition will enable Fuller Co. to offer a full line of dust and fume collection equipment as components of engineered pneumatic conveying systems. Dracco Corp. will operate as an independent subsidiary of Fuller, under the same management.

Nordby appointed program director

Gene M. Nordby has been appointed program director for Engineering Sciences, National Science Foundation, Washington, D. C. He will supervise the foundation's national program for research in engineering sciences.

During the current fiscal year the foundation expects to award grants in excess of a million dollars, primarily to support basic research in civil, chemical, electrical, and mechanical engineering, as well as engineering mechanics and metallurgy, at educational institutions in the United States.

Before coming to the foundation a year ago as an engineer, Mr. Nordby was a member of the civil engineering faculty at Purdue University and at the University of Colorado. Active in ACI projects, Mr. Nordby is chairman of the ACI Committee 215, Fatigue of Concrete, and a member of ACI Committee 115, Research. A paper "Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete," prepared by Mr. Nordby and William J. Venuti, San Jose State College, San Jose, Calif., appeared in the August 1957 issue of the ACI JOURNAL.

Droege-Luther changes firm name

The name of Droege-Luther Testing Laboratory, Fort Wayne, Ind., has been discontinued. The laboratory will be known as the Fort Wayne Testing Laboratory with John P. Luther, owner-manager.

Larson opens engineering office

Marvin A. Larson has opened his own structural engineering office in San Francisco. A licensed structural engineer in California, he was until recently associated with Hall, Pregnoff & Matheu, consulting engineers of San Francisco. He has had both military and civilian experience with the Corps of Engineers, and also has worked with Morrison-Knudsen in Alaska. Mr. Larson joined ACI in 1957.

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Who's Who This Month

Charles S. Whitney

"Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members Without Shear Reinforcement," appearing on p. 265 of this issue, was prepared by Charles S. Whitney, known throughout the world for his contributions to shell and arch design in concrete, and for his work in ultimate load design.

Mr. Whitney, a past president of the Institute, has been an active ACI member since 1940. He twice received the Wason Medal, and in 1951 was accorded the Alfred E. Lindau Award "in recognition of his many contributions to reinforced concrete design practice." Among other honors he has twice received the Fuertes Graduate Medal of Cornell University and the J. James R. Croes Medal from the American Society of Civil Engineers. He is chairman of the Joint ACI-ASCE Committee 326, Shear and Diagonal Tension; chairman of ACI Committee 312, Plain and Reinforced Concrete Arches; and a member of ACI Committee 214, Evaluation of Results of Compression Tests of Field Concrete.

Mr. Whitney is a partner of the firm Ammann and Whitney, New York and Milwaukee, and a graduate of Cornell University.

J. J. Shideler

J. J. Shideler, manager of the Products and Application Development Section, Portland Cement Association, Chicago, is author of "Lightweight-Aggregate Concrete for Structural Use" appearing this month on p. 299.

Mr. Shideler graduated from the University of Denver in 1937 and worked in the materials laboratories of the U. S. Bureau of Reclamation from 1939 to 1943. He then spent 2 years in Alaska working on airport installations for the Civil Aeronautics Administration. Returning to the Bureau of Reclamation he became head of a section in the concrete laboratory responsible for special studies of concrete, concrete cores, grouts and mortars.

In 1953 Mr. Shideler joined the Portland Cement Association and is now manager of Products and Applications Development Section. He is a member of ACI Committee 613, Recommended Practice for Proportioning Concrete Mixes. He has been an ACI member since 1947.

Jack E. Rosenlund

Originally presented at the ACI annual convention in Dallas, Feb. 27, 1957, Jack E. Rosenlund's paper "Construction of the Dallas Memorial Auditorium" now appears on p. 329 of the October JOURNAL.

Mr. Rosenlund graduated from Southern Methodist University, Dallas, in 1944 and then served with the Civil Engineer Corps, U. S. Navy, during World War II. In 1948 he received his MS in civil engineering from the University of Minnesota.

Since 1949 Mr. Rosenlund, an ACI member, has been a structural engineer with George L. Dahl Architects & Engineers of Dallas. In 1952 he became chief structural engineer for the organization.

George C. Ernst

"Ultimate Torsional Properties of Rectangular Reinforced Concrete Beams," prepared by George C. Ernst, professor of civil engineering and director of the engineering experiment station at the University of Nebraska, Lincoln, appears on p. 341.

Professor Ernst, who serves on ACI Committee 115, Research, graduated from the University of Michigan in 1925 and has been in academic work since 1930 at Iowa State College, University of Maryland, and University of Nebraska. In 1956 he resigned from the chairmanship of the civil engineering department at the University of Nebraska to devote full time to teaching and research at the university. He is currently in charge of an investigation of reinforced concrete in

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the plastic range of loading, sponsored by the National Science Foundation, the University Research Council, and the University Foundation.

A previous paper based on this investigation, "Plastic Hinging at the Intersection of Beams and Columns," appeared in the June 1957 JOURNAL.

LOOKING AHEAD

November 5-6, 1957—ACI Regional Meeting, Benjamin Franklin Hotel, Seattle, Wash.

November 13-15, 1957—American Standards Association, Annual Meeting and National Conference on Standards, St. Francis Hotel, San Francisco, Calif.

January 16-18, 1958—National Concrete Products Association of Canada, Annual Convention, Chateau Laurier Hotel, Ottawa, Ont.

February 9-13, 1958—National Ready Mixed Concrete Association, 28th Annual Convention, Chicago, Ill.

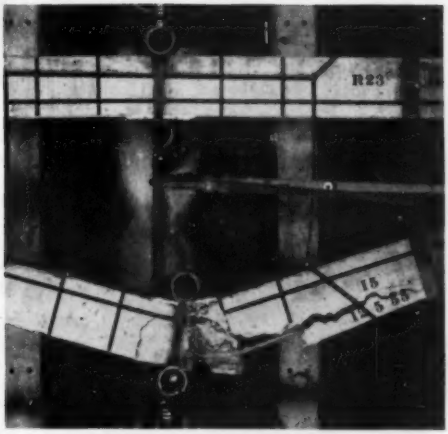
February 17-19, 1958—National Concrete Masonry Association, 38th Annual Convention, Chicago, Ill.

February 17-19, 1958—National Crushed Stone Association, 41st Annual Convention and Biennial Exposition of the Manufacturers Division, Hilton Hotel, Chicago, Ill.

February 24-26, 1958—National Concrete Contractors Association, 1st Annual Convention and 1958 Concrete Exposition, Chicago Coliseum, Chicago, Ill.

February 24-28, 1958—American Society of Civil Engineers, National Convention, Sherman Hotel, Chicago, Ill.

February 25-27, 1958—ACI 54th Annual Convention, Morrison Hotel, Chicago, Ill.



This shot from film, "Destructive Impulse Loads", shows the damage caused by a 40-inch-Kip Spring Energy to reinforced concrete beams.

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"Destructive Impulse Loads" is available free to interested groups of students, structural designers and research engineers.

Other 16 mm color technical films available free are:

"How Concrete Reacts to Near-Blast Loads"	16 minutes
"Advancement in Concrete Pavement"	28 minutes
"Rail Steel in the World of Today"	35 minutes

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*Original report on these tests was made at 1957 meeting of the ASTM.

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Honor Roll

Heading the Honor Roll this month is G. B. Southworth with 16 credits and Blas Lamberti is second with 15 credits. Arthur N. L. Chiu is in third place with 9 credits and H. C. Pfannkuche is fourth with 7 credits.

Since the greatest number of new applicants to the Institute are prompted by persons already on the membership rolls it is up to you, as a member, to help expand YOUR Institute.

G. B. Southworth	16
Blas Lamberti	15
Arthur N. L. Chiu	9
H. C. Pfannkuche	7
Luis Perez Cid	6
Martin J. Gutzwiller	6
Jaime de las Casas	5
Phil M. Ferguson	5
Walter H. Price	5
Nicandro Barboza	4
W. S. Cottingham	4
Louis A. Gottheil	4
Myle J. Holley, Jr.	4
J. E. Jellick	4
Franklin B. Johnson	4
Henry L. Kennedy	4
L. M. Legatski	4
James A. McCarthy	4
Frederick T. Mavis	4
Martin Schulz	4
S. E. Acker	3
James C. Amrhein	3
W. H. Armstrong	3
Luther E. Bell	3
George F. Bishop	3
R. F. Blanks	3
William V. Coyle	3
C. A. Engman, Jr.	3
Ambrosio R. Flores	3
Robert H. Lochow	3
I. Martin	3
George H. Nelson	3
William C. Schwenger	3
Howard Simpson	3
H. T. Williams	3
John T. Young	3
Miles N. Clair	2½
Jose L. Capacete	2
James Chinn	2
Dale Cobb	2
Roger D. de Cossio	2
C. F. De Vilbiss	2
E. I. Fiesenheiser	2
Kenneth H. Gedney	2
Albert Haertlein	2
Roger E. Harlepp	2
N. L. Hinkson	2
Samuel Hobbs	2
M. W. Huggins	2
R. A. Kirkpatrick	2
Leo Liberthson	2
Harry McLean	2

Patrick McNally	2
Ronald A. Maina	2
Izuo Miyashita	2
Otto H. Monch	2
John M. Muir	2
D. F. Nash	2
Richard A. Parmelee	2
Abdur Rahman S. Rasul	2
Theodore O. Reyhner	2
Sabri Sami	2
Wallace Sanders, Jr.	2
Harry Saxe	2
Harold Schweitzer	2
Marcel V. Sous de C.	2
Joaquin Spinel L.	2
J. P. Thompson	2
E. W. Thorson	2
Ellis S. Vieser	2
Joseph J. Waddell	2
D. K. Woodin	2

New Members

The Board of Direction approved 69 Individual applications, 1 Corporation, 20 Juniors, and 6 Students making a total of 96 new members. Considering losses due to deaths, resignations, and nonpayment of dues the total membership on Sept. 1, 1957, was 8974.

Individual

ANDERSON, BRUCE R., Little Rock, Ark. (Arch.)
ARIMA, YOSHIO, Tsuruga-Shi, Japan (Pres., Nippon Prestressed Concrete Co., Ltd.)
BALLANTYNE, JOHN LINDSAY, Montreal, Canada (C. E., E. G. M. Cape & Co. (1956) Ltd.)
BASS, JOHN H., JR., Atlanta, Ga. (Sales Engr., John A. Roebling's Sons Corp.)
BATMANIS, STRATO, Houston, Tex. (Jr. Design Engr., Wyatt C. Hedrick Engrg. Corp.)
BILLIS, STEVE, Staten Island, N. Y. (Design Engr., Michael M. Burris)
BRCEK, S., JAROSLAV, Caracas, Venezuela (Insp. Water lines & Sewers, INOS, Dpto Tecnico)
BROWN, BLAINE, Rothschild, Wis. (Sales Promotion & Adv. Mgr., Chemicals, Marathon Corp.)
BRYANT, WILLIAM C., Temple City, Calif. (Field Engr. Supv., PCA)
BURGESS, E. C., Augusta, S. C. (Contract Supv. in Constr. Div. E. I. duPont de Nemours Co.)
CASTLE, DAVID S., JR., Abilene, Tex. (David S. Castle Co., Arch. & Engrs.)
CHAMBERLAIN, JOHN H., San Francisco, Calif. (Vice-Pres. & Operational Mgr., San Francisco Br., Steelform Contracting Co.)
CHAMBERLAIN, JORGE, JR., San Jose, Costa Rica (Concrete Engrg., Master Bldrs. Co.)
CHERRY, ROBERT J., Royal Oak, Mich. (Dist. Mgr., Euclid Chemical Co.)
CROUSE, ROBERT J., Cleveland, Ohio (Mgr., Cleveland Dist., Pittsburgh Testing Lab.)
DE ANGOLO, FERNANDO, Cali, Colombia (C. E., INCOA, Ltda.)
DE NEYMET, AGUSTIN, Mexico, D. F., Mexico (C. E., Concrete Struct., Mfr. of Concrete Admixtures)
DIYAROGLU, PARESH SIRUN, Ankara, Turkey (Cons. Struct. Engr., Hydroelectric Sect., ILLER Bank)
DRAKE, PAUL E., Omaha, Neb. (Struct. Engr., Corps of Engrs., U. S. Army)
FABER, FRED S., JR., Harrisburg, Pa. (Cons. Engrg. Firm)
FIALKOW, MORRIS N., Jericho, N. Y. (Chf., Design Br., U. S. Army Engr. Dist.)
GORDON, BEN, Alhambra, Calif. (Struct. Engr., Murray Erick Assoc.)

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GREENGARD, CHARLES W., Deerfield, Ill. (C. E., Sanitary Engr., Charles W. Greengard & Assocs.)
GUNTIN, DANIEL A., Brooklyn, N. Y. (Arch. Repres., L. Sonneborn Sons, Inc.)
HART, ALAN S., Marysville, Calif. (Dist. Engr., Div. of Hwys., State of Calif., DPW.)
HART, ROBERT G., Milwaukee, Wis. (Vice-Pres., Siesel Constr. Co.)
HILL, HORACE G., III, Redwood City, Calif. (Chf., Engrg. & Research, Kemic of Northern Calif., Inc.)
HINZE, HERBERT H., Willits, Calif. (Jr. C. E., Constr. Engr., Bridge Dept., Calif. Div. of Hwys.)
HOGE, JOHN H., Cincinnati, Ohio (Sec., Hoge Warren Zimmermann Co.)
HOWEY, A. F., Hamilton, Ont., Canada (Sect. Mgr., Inedible Bulk Products, The Proctor & Gamble Co. of Canada, Ltd.)
HUSTED, HAROLD B., Youngstown, Ohio (Cons. Civil & Struct. Engr.)
ISAVA CARBONELL, MANUEL ALFREDO, Caracas, Venezuela (C. E., Ministry of Pub. Wks. of Venezuela)
JACOB, PAUL, Trois-Rivieres, Que., Canada (Vice-Pres. & Gen. Mgr., Laurentide Ready Mix & Equipment, Inc.)
JACOBSON, GUSTAF W., Worcester, Mass. (Arch. Design & Struct. Engrg., E. J. Cross Co.)
JONES, J. E., Calgary, Alta., Canada (Sales & Dev. Engr., Alberta Ytong Mfg. Co., Ltd.)
KE-HUA, HOU, Singapore, Malaya (Asst. Engr., Hume Industries (Far East) Ltd.)
KENNEDY, FRANK, 45 Meadow St., Ardwick, Manchester, England (Asst. Engr., Matthew & Mumby, Ltd.)
KENNISON, H. F., East Orange, N. J. (Vice-Pres., Engrg. & Research, Lock Joint Pipe Co.)
LA BRIE, ROBERT JAMES, Monterey Pk., Calif. (Asst. Struct. Engr., State of Calif.)
LAPUK, ROBERT A., Hartford, Conn. (Struct. Engr., A. J. Macchi)
MAAS, EDWARD A., JR., Allentown, Pa. (Office Engr., Lehigh Portland Cement Co.)
MACFARLANE, GEORGE, Cape Coast, Ghana (Tech. Adviser, Dept. of Social Welfare & Community Dev.)
MEAD, PHILLIP R., Sioux Falls, S. D. (Struct. Engr., Hugill, Blatherwick, Fritzel & Kroeger, Archs.)
MITCHELL, NEAL B., Salem, Mass. (Asst. City Engr., City of Salem)
MORVAY, ANTON A., Elmwood Pk., Ill. (Struct. Engr., A. J. Boynton & Co.)
NEGUS, CHARLES E., JR., Auburn, Calif. (Assoc. Bridge Engr., State of Calif., Div. of Hwys.)
NUNN, H. JACK, Atlanta, Ga. (Div. Mgr., The Master Builders Co.)
OCHALEK, RICHARD T., San Francisco, Calif. (C. E., Adrian Wilson & Assocs., Archs., Engrs.)
O'FALLON, JOHN D., West Hartford, Conn. (Div. Bridge Engr., Bur. of Pub. Rds.)
PEREIRA, LUIS A., Canton, Ohio (Designing & Sales Engr., Export Dept., Macomber Panamericana)
PETERS, JACK S., Hurricane, W. Va. (Sales & Service, Master Builders Co.)
POCZA, LESTER, Milford, N. J. (Contr. Bldr.)
RAJU, ALAMENGADA A., Chicago, Ill., (Struct. Engr., Skidmore, Owings & Merrill)
REVAY, STEPHEN, Montreal, Que., Canada (Chf. Engr., Dufresne Engrg. Co., Ltd.)
RUP COSSON, EDMUNDO ALBERTO, Caracas, Venezuela (C. E., Hydraulics Dept., Ministry of Pub. Wks.)
RUSSELL, THOMAS O., JR., Fullerton, Calif. (Concrete Field Engr., Super Concrete Emulsions, Ltd.)

SCHEUER, LOUIS, Salisbury, Md. (Struct. Engr., Salisbury Steel Products, Inc.)
SCHOENING, W. H., Manila, Philippines (Vice-Pres. in chge. Constr., Atlantic, Gulf & Pacific Co. of Manila)
SIEBENTRITT, HAROLD F., Chicago, Ill. (Pres., Standard Painters, Inc.)
SMITH, JOHN H. H., Saguenay, Que., Canada (Field Engr., Anglin-Atlas Ltd.)
SMITH, VERNON H., Memphis, Tenn. (Bridge & Struct. Engr.)
TOWNSEND, CHARLES L., Denver, Colo. (C. E. USBR)
TYGE, KNUD, Detroit, Mich. (Struct. Engr., Giffels & Vallet, Inc.)
VAN OYEN, H. W., Glenbawndam via Scone, N. S. W., Australia (C. E., Constr. Br., Water Conservation & Irrigation Comm.)
VELTROP, JAN ADRIANUS, Lombard, Ill. (C. E., Harza Engrg. Co.)
VILLENA, GUILLERMO, Cincinnati, Ohio (Proj. Engr., Alfred LeFeber & Assoc.)
YEAGER, GILBERT H., El Paso, Tex. (Engr. in chge. Constr. Projs., Robert E. McKee, Gen. Contr. Inc.)
ZIMMERMAN, K. E., Houston, Tex. (Chf. Engr., Walter P. Moore, Cons. Engr.)

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BALDWIN, DONALD BROOKS, Louisville, Ky. (C. E. Trainee, U. S. Army Corps of Engrs., Louisville Dist. Office)
BENNETT, J. D., London, England
BENSON, 1/Lt. DANA A., Rome, N. Y. (Struct. Engr., USAF, Air Research & Dev. Command, Rome Air Dev. Center)
BUCHER, K., HANS R. E., Tome, Chile (Bldg. & Constr. Engr., Fabrica de Panos Bellavista)
ELSESSER, ERIC, San Francisco, Calif. (Struct. Designer, John A. Blume & Assocs.)
ERDMENGER L., JORGE ERNESTO, Guatemala City, Guatemala (C. E., Bridge Dept., Design Sect., Hwy. Dept. of Guatemala)
GIBSON, JOHN BARRY, Nedlands, Western Australia (C. E., Pub. Wks. Dept., Hydraulic Br.)
HALE, JOHN S., JR., Lewisburg, Pa. (Instructor, C. E. Dept., Bucknell Univ.)
HARDAWAY, LEMUEL HALL, JR., Nashville, Tenn. (Est., Hardaway Constr. Co.)
ISIDORO, JUAN B., Quezon City, Philippines (C. E., A. Flores, Engr.)
JONES, KENNETH, Gary, Ind. (Engr. Recruit, American Bridge Co.)
LONGANECKER, GERALD W., Hyattsville, Md. (Design Engr., Nuclear Products, ERCO Div., ACF Industries, Inc.)
LOTTMANN E., JOAQUIN, Guatemala, Guatemala (C. E., Designer, Direcc. Gral. Caminos Seccion Ouentes)
MOUTON, WILLIAM JOSEPH, JR., Harahan, La. (Design Engr., Walter E. Blessey, Cons. Engr.)
PANICARI, CARLO F., Reno, Nev. (Constr. Engr., McKenzie Constr., Inc.)
RODRIGUEZ, AUGUSTO E., New York, N. Y. (Jr. Designer, Ammann & Whitney, Cons. Engrs.)
RUIZ RAMIREZ, GILBERTO A., Chihuahua, Chih, Mexico (C. E.)
WEATHERS, ISAAC BRANTLEY, JR., Colorado Springs, Colo. (Engr., Farnsworth & Chambers)
WONG, WILLIAM LEE, New York, N. Y. (Struct. Engr., Ammann & Whitney, Cons. Engrs.)

Student

EMIROGLU, MEHMET HALUK, Lawrence, Kans. (Grad. Stud. Univ. of Kansas)
FIGOL, ERNEST PETER, Edmonton, Alta., Canada (Univ. of Alta.)
JACOBS, H. MARTIN, Orange Lake, Newburgh, N. Y. (Engr. in Trng., Burns & Roe, Inc.)
JUST, J. T. H., Wellington, New Zealand (Canterbury Univ. College)
NASR, GEORGE, Fritz Lab., Bethlehem, Pa. (Lehigh Univ.)
TUEN, GORDON L., Austin, Tex. (Univ. of Tex. & Draftsman, Texas Hwy. Dept.)

Southern Cement expands

Southern Cement Co., a division of American-Marietta Co., has begun operations at another major cement production unit in Roberta, Ala. Facilities have also been completed there increasing output of chemical lime by 60 percent. Installation of a 420 ft long dry process rotary kiln, together with related equipment increases Southern's annual cement capacity by 1½ million barrels.

Established in 1901, Southern's original cement plant is at North Birmingham, Ala. The company owns close to 1000 acres of proved limestone reserves adequate for 70 years' operation.

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

(cut here)

Board of Direction, American Concrete Institute
P. O. Box 4754, Redford Station
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Date_____

Individual Members (U. S. and Possessions, Canada, Mexico, Central America, and West Indies)	$15.00
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Corporation Members	50.00
Contributing Members	100.00
Junior Members—nonvoting (under 28)	7.50
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(Subject to stipulations of Bylaws—Article I on reverse side. Bylaws on request.)

Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for_____ membership
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by_____

For Corporation Membership, ACI representative will be_____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature_____

For our records, please complete both sides of application.

EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Members, Corporation Members, Contributing Members, Junior Members, Student Members, and Honorary Members interested in furthering the Institute's objects as set forth in its Charter.

Sec. 2. A Member shall be an individual. A Corporation Member shall be a firm, corporation, society, agency of government or other organization. A Corporation Member shall name one individual as its representative who will enjoy all membership rights and privileges.

A Contributing Member shall be an individual, firm, corporation, society, agency of government, or other organization wishing to give larger support to Institute activities through the payment of larger dues. Contributing Members, other than individuals, shall name representatives as do Corporation Members.

A Junior Member shall be a person less than 28 years old.

A Student Member shall be less than 28 years old and a student in residence at a recognized technical or engineering school.

An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinarily meritorious service to the Institute.

Sec. 3. All classes of Members except Honorary Members and Student Members, shall be proposed by at least one Member of the Institute and elected by a two-thirds vote of the Board of Direction. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be proposed by a member of his school's faculty, who need not be a Member of the Institute, and elected by a two-thirds vote of the Board of Direction.

Sec. 4. All Members shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall not vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member on the first anniversary of his membership succeeding the date on which he ceases to be a student in residence. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 5. Applications for and resignations from Membership and requests for change of representatives of Corporation or Contributing Memberships shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

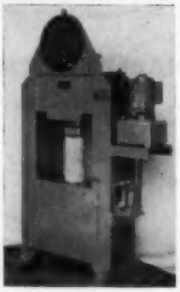
Anchor catalog

Catalog No. 57 showing the complete line of Bulldog expansion anchors and pin bolt drives is now available. It lists five types of self drilling expansion anchors: special flush anchors, rod hanger anchors, tie wire anchors, stud head anchors (installation for all four is with electric or air hammer), and flush anchor (installed by hand hammer only). Catalog also gives complete details on three types of Bulldog pin bolt drives: round (removable), or permanent flat head for finish work, and stud head type. Also shown are flush anchor holders, wire ceiling plates, hammer chucks and toggle bolts.—J. D. Polis Manufacturing Co., 2900 W. 26th St., Dept. A 169, Chicago 23, Ill.

Investigation of masonry wall ties

Armour Research Foundation of Illinois Institute of Technology, has just completed some studies on the possibility of eliminating the troublesome header course in two wythe masonry wall construction. These studies were sponsored by AA Wire Products Co., Chicago, who manufacture ECONO-LOK wire tie, a masonry reinforcement designed to control shrinkage cracking in concrete masonry walls. According to the manufacturer ECONO-LOK was designed as a combination reinforcement and tie which tends to control the difference in the coefficient of expansion and contraction between the most common type of two wythe wall, namely concrete block backup and face brick facing. The masonry tie provides two parallel reinforcing wires which are designed to rest on the face shell webbing of the concrete block backup. Flush welded to these parallel reinforcing wires are 4-in. wide rectangular ties every 16 in. which tie the backing to the facing.

Conclusions of the investigation reveal that replacement of header courses by wire reinforcement does not reduce the transverse strength of a wall; on the contrary, it may increase it. Also, replacement of header courses by wire reinforcement does not reduce the compressive strength of a wall, and wire reinforcement of correct design can satisfactorily replace header courses. —AA Wire Products Co., 714 East 61st Street, Chicago 37, Ill.



Testing machine for concrete

Designed for testing high-strength concretes, the Model QC-125-LD compression tester with a capacity of 125 tons is described in the manufacturer's Bulletin QC-125-LD. The machine can test 6- x 12-in. cylinders up to 8850 psi, and 8- x 8- x 16-in. block up to 2100 psi. Equipped with constant flow pump adjusted to load between 20 and 50 psi per second, machine offers uniformity of loading within the range of ASTM and AASHO specifications, according to manufacturer. The gage is mounted on springs to absorb the shock from breaking specimens.—Forney's Inc., Tester Division, Box 310, New Castle, Pa.

Bulletin on laboratory ovens

Soiltest, Inc., now has Bulletin 6-57 on special laboratory ovens available. Humidity cabinets and water baths are also covered in the illustrated bulletin which gives inside dimensions of the chambers. Manufacturer advises that equipment has been designed to meet ASTM and AASHO specifications for accuracy and is suited for use in either the laboratory or on construction projects.—Soiltest, Inc., 4711 W. North Ave., Chicago 39 Ill.

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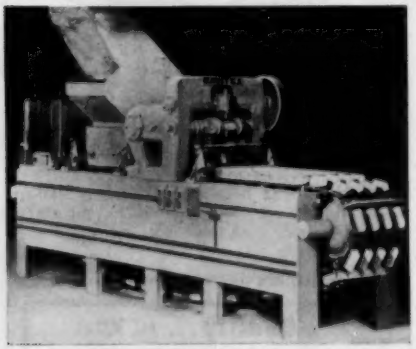
The Institute assumes no responsibility for the claims of advertisers. The advertiser is made responsible in the belief that his place in the field will be determined by the public's ultimate measure of his exercise of that responsibility.

Compact block production machine

An endless-chain machine is available for local manufacture of double-cavity concrete block. The machine receives a precision-measured mix, then tamps, trowels,



and forms it into the Cavitex units with either striated or plain texture. Face of the unit is seven times as long as it is high, and block are either 4 or 8 in. wide. With ⅜-in. mortar joints, the length of each unit in the wall is 16 in., and the height 8 in. to every three courses. Manufacturer believes these masonry units can be produced at prices competitive with lumber.—W. E. Dunn Mfg. Co., 302 W. 24th St., Holland, Mich.

Waterstops

Recently issued catalog sheet of waterstops provides a convenient table listing the recommended joint application, recommended head for each size and kind of waterstop. Detail drawings show Labryinth, Flextrip, cellular, and dumbbell waterstops in their various sizes. According to the manufacturer, Water Seals made of flexible polyvinyl plastic resist chemical and weathering changes through the years, and form a lasting closure even in joints where extreme movement is anticipated. —Water Seals, Inc., 9 South Clinton, Chicago 6, Ill.

Automatic clutch on power trowel

Belts were eliminated on the T-29 power trowel through use of a Mercury automatic clutch. This trowel model uses a 2-hp Briggs and Stratton, 4-cycle, air-cooled, vertical shaft engine. With the clutch preset at 1800 rpm, the speed reduction gear train is automatically engaged when the engine reaches that speed. Sudden motor strain or stalling when blades strike unnoticed or protruding objects are prevented since the clutch releases completely when the engine slows below minimum operating speed. Engine speed ranges up to 3600 rpm.—Thor Power Tool Co., Chicago Ill.

Reinforced Concrete Design

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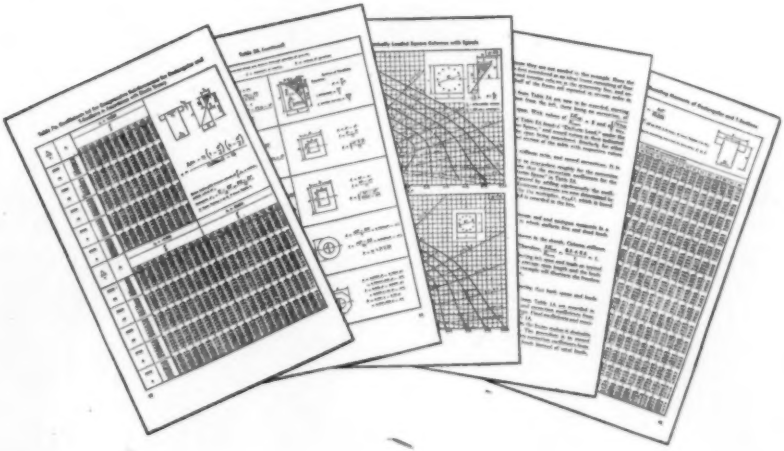

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